August 17, 1933

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New Orleans-Mississippi River Bridge

Bristol Power Dam Raised With Plant in Operation

Fire Tests of
Gypsum-Incased Steel Columns

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Survey of Traffic Volume
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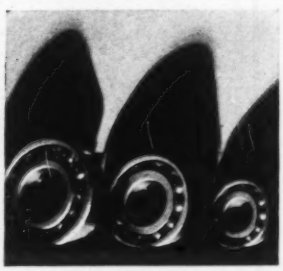

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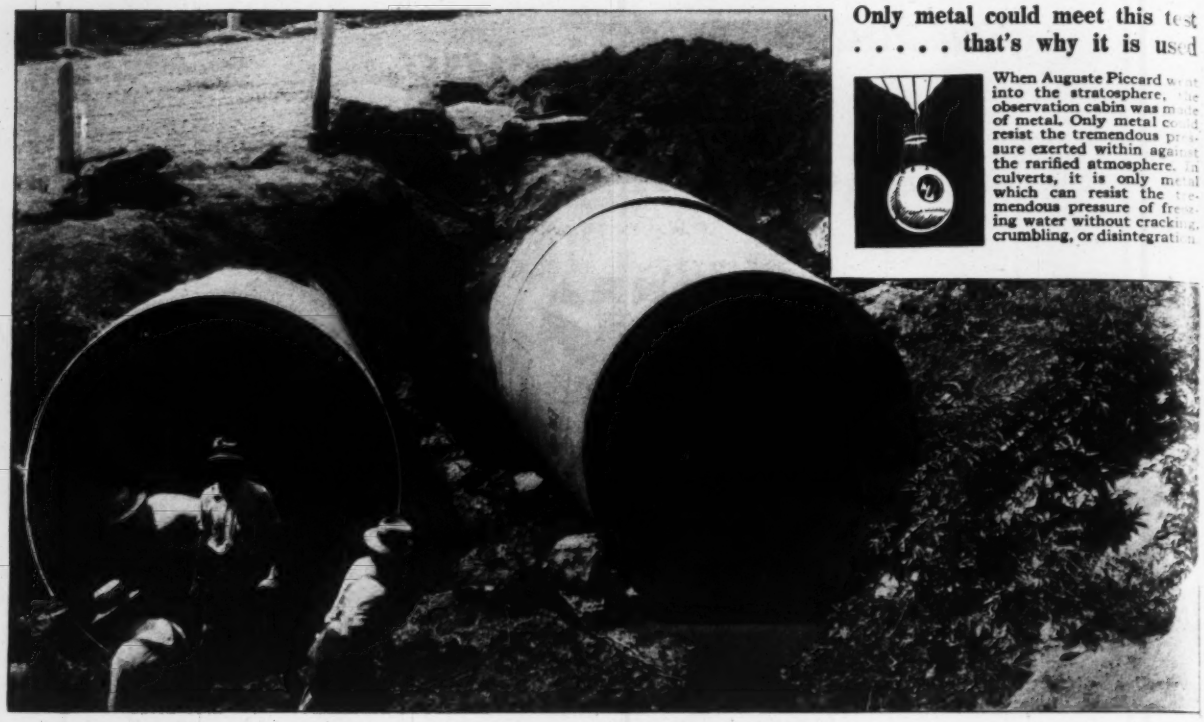

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Power Dam Raised 30 Ft. Under Full Head

Old crest cut down without taking power plant out of service—Extended concrete cutoff wall built from top down to depth of 110 ft.—Hinged steel flashboards installed

CAREFUL advance designing and planning of the initial construction, as described in *Engineering News-Record*, Aug. 18, 1927, and Feb. 25, 1932, made it possible to increase the height of the Bristol Dam and power house by 30 ft. The head on the plant was increased from 50 to 80 ft. by this construction without a shutdown of the power plant. Aside from the problems encountered in raising the dam, other interesting features were a deep cutoff wall that was carried to ledge rock through 110 ft. of gravel, clay and boulders without the use of compressed air, the provisions for the addition of a penstock for the third stage of power development, and the installation of 8-ft.-high hinged structural-steel flashboard panels on the spillway crest. These panels are exceptionally easy to handle for a crest - regulating device of this size.

The Bristol Dam is located on the Pemigewasset River near Bristol, N. H. The dam as initially constructed in 1923 consisted of 300 ft. of spillway and 225 ft. of bulkhead and power house. The construction of the extension, which raised the head of the dam 30 ft., increased the over-all length of the dam to 650 ft. The total flood-discharge capacity of the structure is 120,000 sec.-ft. The maximum flood handled since the original structure was built is 60,000 sec.-ft. in the severe New England floods of 1927.

Raising the dam

The 30-ft. increase in height of the spillway and the power-house intake were designed in the general form of inclined concrete columns superimposed upon and forming extensions to the initial buttress construction. Fig. 1 shows the completed project after the initial structure had been raised 30 ft. In planning the initial construction and future extension during 1921 and 1922, care was taken to see that the joint between the two sections coincided approximately with a trajectory of principal stress. This provision both minimized the shear along the joint between the two sections and made each section independently stable under the ultimate head on the dam. After determining the correct plane between the initial and final construction, each section was independently checked for stability under the final head.

Fig. 1—Bristol Dam in New Hampshire after raising head 30 ft. View shows hinged-steel flashboards and cableways for raising and lowering them.

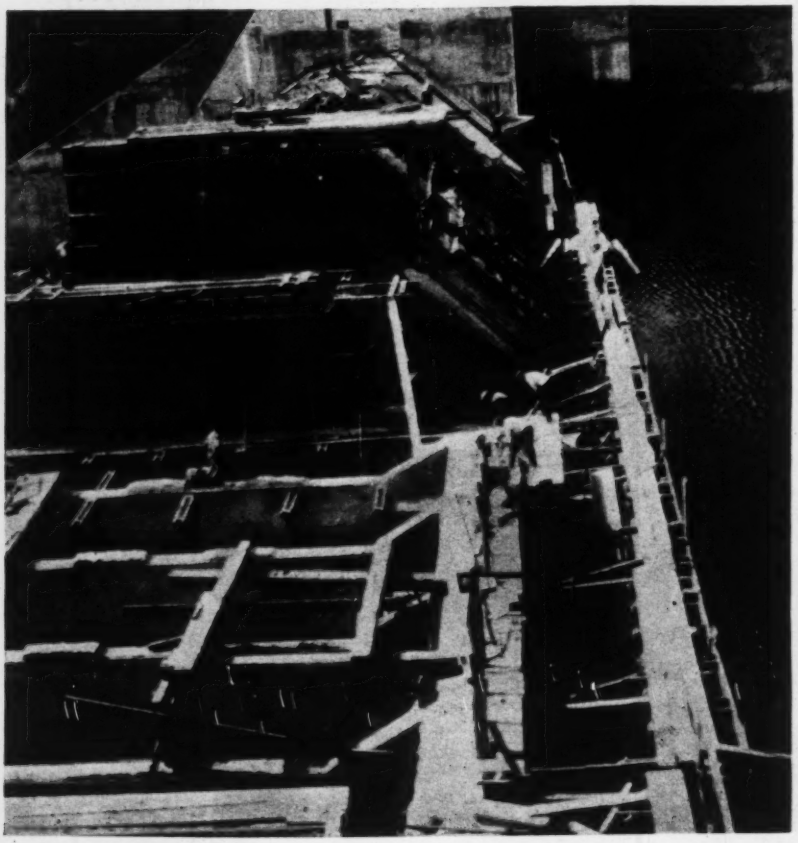
To connect properly the initial construction as planned in advance and the extension, it was necessary to remove the 2-ft.-thick spillway crest slab 3 ft. from the top of the existing spillway buttresses and the existing spillway apron. While the removal of this masonry was in process the water was kept at the crest level and the full normal operating head was maintained at all times by means of movable floating timber coffer-

dams constructed in sections 15 ft. long (Fig. 2). These were weighted down with steel bars and bolted to the upstream face of the deck. Old canvas belting was used to seal all joints where the sections of this cofferdam came in contact with the concrete. The small amount of water that did leak into the working space was siphoned out by suction hose.

The original rollway apron or downstream-face slab of the initial spillway was removed by cutting it into sections or blocks with paving breakers, as shown by Fig. 3. These blocks or sections were either pulled away by cables or were allowed to fall between the buttresses.

Care was taken to bond together properly the initial construction and extension both by roughening the old concrete and by doweling the sections together, although such bonding was not actually required to insure the safety of the dam owing to the fact that both the original and the new sections were designed as independent structural elements. In both the intake and the spillway, the dowels were originally embedded in the downstream face of the initial construction. One section of each dowel was placed a few inches below the surface and was later chipped out and straightened to form a tie after the buttress faces had been roughened preparatory to bonding the extension to the initial construction.

Fig. 2—Movable timber-panel, limpet-type cofferdams for removing crest of old dam without lowering head of water.



Cutoff walls

Identical types of cutoff walls were constructed adjacent to the abutments in both the east and the west banks, although the most difficult construction problems were encountered in the east bank on the power-house side of the river where the wall is carried to a maximum depth of 110 ft. Ledge rock east of the power-house slopes away from the ground line (as shown by the elevation of the wall on Fig. 4) instead of following the more usual course of sloping upward in the general direction of the earth contours above. In order to insure adequate resistance against percolation, a wall was therefore required that increased in depth eastwardly until the foundation was in ledge rock about 110 ft. below the ground surface. Beyond this point the foundation wall was in clay and hardpan and stepped upwardly to a minimum 30-ft. depth below the ground line at the end.

To construct a masonry wall of these proportions by either the conventional open-trench or the caisson method would have been extremely costly and also quite hazardous. Comparative designs and estimates for several types of cutoff walls indicated that the cellular-wedge type recently developed by the Ambursen Construction Co. for the Rodriguez Dam in Mexico (*Engineering News-Record*, Oct. 16, 1930) would be the most feasible and economical to construct. The experience gained in carrying out other deep cutoff wall construction of this type served to good advantage on the Bristol job, although the problems caused by the widely varying materials encountered in the Bristol corewall excavation called for ingenuity and skill on the part of the constructors.

In constructing this type of wall, first an open trench, about 14 ft. wide and 16 ft. deep, was excavated for the length of the first concrete section to be poured. The sides of this trench were roughly sloped as shown by Fig. 4 so that the cellular masonry section subsequently poured would be supported by wedge action when the excavation for a second cellular-wedge section was carried out below. Both the east and the west cutoff walls were constructed in this manner by consecutive downward steps until

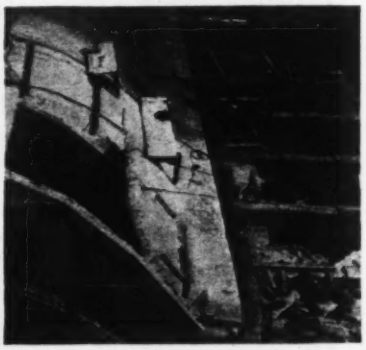


Fig. 3—Method of removing apron slab from top part of original dam.

bedrock was reached. This created a series of cellular wedge-shaped concrete sections (Fig. 4) that were tied together with continuous vertical and horizontal steel. When the bedrock was finally reached, a trench 3 or 4 ft. deep was channeled into it as shown by Fig. 4, and this bottom section of the corewall was poured as a solid block up to the bottom of the suspended cellular section next above.

Provision was made to grout all vertical and horizontal joints between the concrete pours. These grout pipes, located in the wall, served two functions: first, as drains for the purpose of relieving hydrostatic pressure in the sides of the wall during construction and taking off all seepage to a convenient sump before it reached the workings at lower levels; and second, for the purpose of making the wall completely watertight by the grouting process after construction had been finished.

When these joints were grouted at the completion of the cutoff-wall construction, a surprisingly small amount of grout was required, the joints themselves taking practically none. Some of the pipes that had been extended through

the sides of the cells to serve as drains during construction took a few bags of cement; the greatest quantity, however, was usually through pipes that were installed for the express purpose of subsequently refilling with grout, areas where the gravel or sand had sloughed downward from behind the wall during the construction of the cellular unit next below.

After the concreting and grouting were completed, the cellular spaces were backfilled with selected earth, and the top of the wall was capped with a low solid gravity section. Fig. 5 shows the top of the wall before this gravity section was placed.

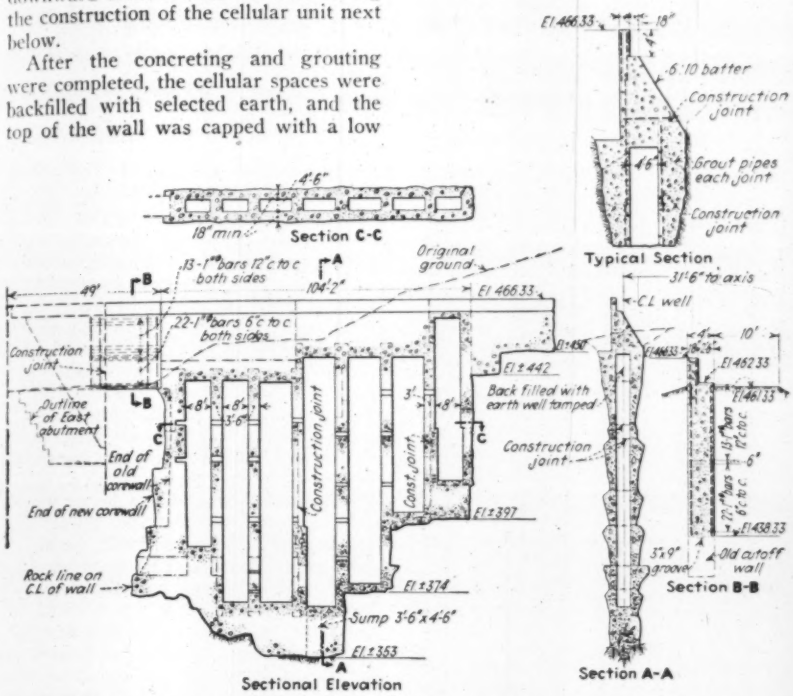


Fig. 4—Details of cellular concrete cutoff walls constructed from top down and capped with gravity section above ground.

The remarkable feature of this cutoff or corewall construction was the speed with which it was completed, a total period of only 95 days being required for all this work on both sides of the river. The wall on the east or power-house side was 120 ft. long, with a maximum depth of 110 ft., and the west side wall had a length of 50 ft. and a depth of 60 ft.

Careful attention had to be given to the design of the cutoff wall in order to provide for the varying pressures caused by the widely different classes of foundation materials that were encountered. Not only was it necessary to change the thicknesses and reinforcement of the diaphragm walls to take care of this pressure variation; it was also necessary to study the characteristics of the various classes of foundation material under pressure so that sufficient wedge action could be provided to insure adequate support for the cellular-wedge sections.

Where the softer soil was encountered, great care had to be exercised in order to prevent the material at the sides from starting to squeeze and either slough downward to fill up the excavation below the cellular sections or crush the cells themselves by a general soil movement. When there was evidence that this might occur, excavation was discontinued temporarily in that area and pressed ahead rapidly in the firmer soil on each side of the threatened shaft. Those parts of the wall constructed in firm material thus supported the cellular-wedge sections in the softer material while the wall was being rigidly tied together with reinforcement and each section well keyed into the adjacent blocks. With the pressure from the weight of the wall thus removed from the softer or wetter areas, excavation could again proceed therein with safety. This required very careful advance study of the texture and water-bearing characteristics of the material at all points, and the entire operation required great care and skill in performance, although it was rapidly carried through with no injury to any workman and with no displacement of material adjacent to the wall, as proved by the very slight amount of grout that could subsequently be injected to refill such disturbed areas.

Future penstock bay

Between the gravity abutment at which the east bank corewall terminates and the power-house are two open bulkhead bays. The bulkhead bay adjacent to the power-house was modified to support a 10-ft.-diameter steel penstock that is to be installed at a later date. This provision puts the Bristol project in the unusual class of a three-stage power development, using both the methods of obtaining further increased capacity by providing for the addition of power units and by raising the dam. Provision has been made so that the future rebuilding of the intake will also be carried on without interfering with the operation of the plant.

Spillway flashboards

A relatively new type of flashboard (Fig. 6), which has been successfully used in connection with several other dams constructed for the New England Public Service, the parent company of the Public Service Company of New Hampshire, was installed to a height of 8 ft. on the crest of the spillway extension. These gates consisted of welded structural leaf sections, 5½ ft. wide, which are hinged at the crest. Each steel leaf is supported by a 4x4-in. wooden strut that rests in a notch in the crest.

Provision has been made to operate the gate by two methods. The first of these methods provides for emergency operation, which consists of pulling out the supporting struts by means of cables that are strung through these struts and carried over sheaves at the end piers to winches located on the tops of these piers. Two cables are used, half of the flashboards being controlled by each winch. In dropping the flashboards by this method the cables are drawn in by the winches, and a knot in the cable end engages the center strut and breaks it, thus lowering the first board. The cable is then drawn in until the second strut is engaged and broken, and so on until all the crest gates are released. Experience has shown that the fall of the leaf is well cushioned by both water and air and that no damage results by dropping the gates in this manner.

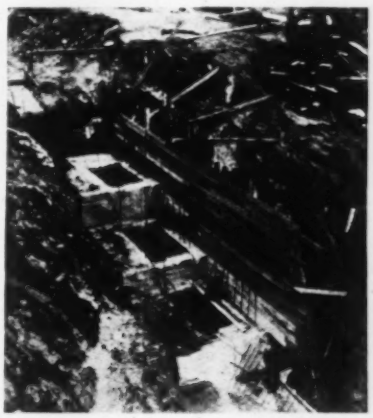


Fig. 5—Top of cellular concrete cutoff wall, showing some of the cells backfilled with earth.

The second method of controlling the flashboards, the normal operation, is both to raise and lower them by a hook that is operated from a pair of perma-

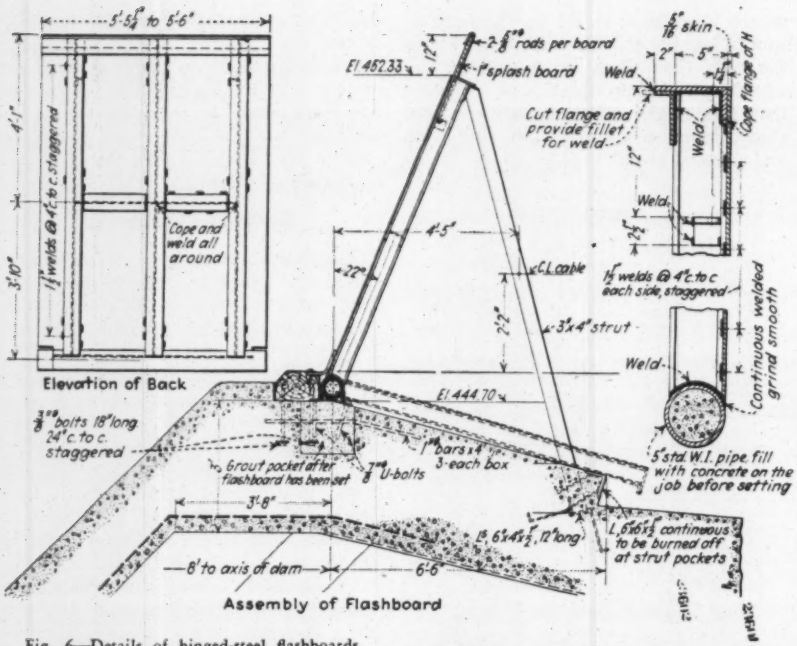

Elevation of Back
Assembly of Flashboard
Cope and weld all around
Weld
Cut flange and provide fillet for weld
Continuous welded grind smooth
3'x4" strut
5' std. W.I. pipe fill with concrete on the job before setting
6 grout pocket after flashboard has been set
U-bolts
bars
3-each box
8'to axis of dam
6'-6"
splash board
rods per board
skin
Cope flange of H
C.L. Cable
welds @ 4'c. to c. each side, staggered
bolts 18" long 24" c. to c. staggered
L 6"x6"x½" continuous to be burned off at strut pockets
El 462.33
El 444.70

Fig. 6—Details of hinged-steel flashboards for raising the spillway crest 8 ft.

nent flashboard-operating cableways over the spillway structure. The operations of raising and lowering the flashboards and of placing the struts are carried on from a man-car suspended from a second cableway a short distance downstream from the main operating cableway. This second method of operating the boards will undoubtedly be used more frequently than the first method.

After completion of the dam a test of these unusually large flashboards was made under full 8-ft. head, three panels being lowered in 6 min. and raised against full head in 9 min. At this rate the 48 sections could be lowered under flood conditions in about 1½ hours, and under similar conditions could be raised in less than 2½ hours.

Construction methods

The original construction in 1924 exhausted the near-by sources of sand and gravel for concrete aggregates, so that it was necessary to utilize a shallow river deposit 2 miles below the dam. The construction work incidental to raising the height of the dam extended from the summer of 1930 over into 1931, with a winter shutdown of about 3½ months required by the Public Service Commission of New Hampshire. As this gravel pit was submerged during the high water of spring and early summer, it was necessary to haul practically the entire requirements of sand and gravel during the fall of 1930 and store the run-of-bank materials in large stockpiles at the site of the work.

Power shovels were used for digging and loading the material at the pit. A small speeder shovel reclaimed it at the stockpiles, loading into small trucks that dumped the unwashed raw materials direct into the receiving hopper for the bucket elevator at the principal storage bin. Oversize material was passed into a crusher whose output fed back into the main bucket conveyor. Everything was elevated to the top of the main bin where the material was washed and classified. A large semi-portable steel-frame secondary bin equipped with measuring hoppers with a 1-yd. mixer below is fed from the larger timber-storage bunkers with bucket elevators, making an installation that required very few men for economical operation.

The principal equipment consisted of cableways. A large electric cableway with 2½-in. main cable on 80-ft. timber A-frames was placed directly over the axis of the dam. A secondary cableway with a 1¾-in. cable on 50-ft. A-frames was installed 25 ft. farther downstream, thus fully covering the reconstruction work. Concrete was placed by means of 1-yd. bottom-dump buckets handled by the cableways direct from the mixer to the forms with no chuting or rehandling. For the long downstream buttress extensions these buckets were transferred to car frames on sectional track placed directly on the top extensions of large sturdy form panels, thus avoiding all manipulation of the concrete in the forms except the usual spading or rodding and eliminating any tendency to separation of aggregates.

The large form panels as well as all reinforcing steel were placed with the cableways, and much of the excavation was also handled with this equipment.

A stiff-leg derrick was set up at each abutment to handle the muck from the corewall excavation as well as from the extensions to the dam abutments themselves, and a caterpillar crane served as general utility equipment.

The project was constructed for the Public Service Company of New Hampshire, of which F. H. Mason is chief engineer and A. R. Schiller is general manager. The initial and final construction of the dam and power-house were both designed and built by the Ambursen Construction Co., of New York. John L. Frazier was superintendent in charge of construction, and H. D. Resseguie served as resident engineer representing the owner.

Handling Smoke Abatement

PROGRESS in reducing city smoke nuisance and damage is being accomplished largely by extensive use of educational methods, according to discussions at the American Society of Mechanical Engineers' Chicago convention, June 26-30. The discussion indicated that numerous technical problems still await solution.

William G. Christy, smoke abatement engineer for Hudson County, N. J., reported success in dealing with the smoke from shipping in New York harbor. Education in proper combustion and information as to the economic losses due to smoke are better means of solving the problem than punishment for the production of smoke, he stated. Makers of boilers and furnaces still pay too little attention to the matter of how their product is to be operated and what fuel is to be used. For example, equipment designed for anthracite is being sold for use with bituminous coal.

Thorough attention needs to be given to the human element, since the best equipment will produce smoke if not properly operated, and the fireman's care depends in part on the attitude of his superiors. While smoke abatement essentially is an engineering problem, political interference often hampers enforcement, and there is the difficulty of making the public and plant managers "smoke-conscious."

These ideas were reflected in the discussion, but it was pointed out that in many cases smoke campaigns are short-lived, due either to direct opposition or to incompetent political management. In Chicago success is being attained through educating firemen in the proper handling of boilers and informing plant managers as to its economic value, according to Frank Chambers, city smoke inspector. Legal action is reserved for dealing with direct opposition.

Methods of measuring cinders (from stokers) and fly-ash (from pulverized coal) were outlined by A. C. Stern, of Stevens Institute of Technology. This study was started because of agitation for legal limits of permissible emission of cinders and fly-ash. Sulphur compounds in waste gases are also a source of trouble, however, as was pointed out by H. F. Johnson, of the University of Illinois. Methods for removing these compounds from the flue gases are available.

Foundations Get Under Way on New Orleans Bridge

Special open caissons, to be sunk 180 ft. through 90 ft. of water by the sand-island method, have closely spaced wells in the concrete walls that provide access to the cutting edges —Willow mattresses sunk at each pier to prevent scour

THE foundation contractor on the combined railway and highway bridge over the Mississippi River, 10 miles above the "old city" section of New Orleans, has initiated his work by weaving and sinking large willow mattresses over each of the river pier locations. These mattresses, four of which are now finished, are required to prevent scour as the caissons are being sunk. That such scour is a distinct possibility was revealed during the taking of test cores, when holes 12 to 14 ft. deep occurred around the 4-in. drill casings. Under normal conditions the river current has a velocity of 4 miles per hour. With the mat work completed, caisson work will be started. The deepest of these foundations will be sunk 180 ft. below mean gulf level, or about 195 ft. below the maximum working stage. It is planned to do this work without resorting to compressed air, as four of the caissons will be sunk far below limits for pneumatic work, and unusual open-bottom caissons have been designed for the purpose.

The New Orleans bridge is being built jointly by the Public Belt Railroad Commission and the state of Louisiana at an approximate cost of $13,000,000, loaned by the Reconstruction Finance Corporation. Upon completion the bridge will be used by the Southern Pacific Railroad in addition to the Public Belt Railroad and the main state highway into New Orleans from the northwest. The bridge and approaches have a total length of 23,000 ft., the river crossing being a cantilever structure with a 790-ft. main span and 530-ft. anchor arms. The remainder of the main bridge superstructure consists of a 530-ft. through span and four deck-truss spans varying in length from 333 to 269 ft. This span arrangement was made necessary to meet navigation requirements. The long viaducts will have tower spans of 45 ft. and intermediate spans varying in length from about 69 to 82 ft.

A vertical clearance of 135 ft. above mean high water is provided for the main bridge structure. The approach structures will be constructed with the railroad decks on descending grades of 1.25 per cent. The vehicle roadway, however, will descend on a 4 per cent grade. There are two of these vehicle roadways, each 18 ft. wide, supported outside the through trusses on cantilever extensions of the floor beams. On the approaches the roadways are supported on brackets cantilevered from the vertical members of the deck-truss spans and from the columns of the viaduct towers.

Foundation design

Nine piers will support the main bridge, two on pile foundations on the river side of the east levee, which is a "making" bank, one on piles behind the east levee, one caisson pier in the comparatively shallow water near the east bank, four caissons in the deeper portion of the river and one caisson behind the levee on the west bank. Any future shifting of the river channel would probably be toward the west, which is a "cutting" bank, and this accounts for the use of a caisson instead of a pile foundation behind the west levee.

The caissons planned for the bridge are of unusual design and will be open-dredged to the unprecedented depth of 180 ft. below gulf level. Until recently the deepest caisson was at Hawkesbury, Australia, open dredged to a total depth of 162 ft. below water level. During June of this year a caisson for the Atchafalaya River bridge at Morgan City, La., was sunk 176.5 ft., and one of the Trans-Bay Bridge piers at San Francisco will go down 226 ft.

The main caissons must be dredged through the semifluid silt forming the river bottom, and then through alternate layers of sand, clay and gumbo, finally landing on a compacted sand. In the design of the caissons two opposing factors required consideration: first, the unprecedented depth to which they must be sunk (through material that may offer great frictional resistance) requires a maximum weight during sinking; second, because of limitations in expected long-time settlement of the piers under load the maximum soil pressure on the compact sand at El. —170 was established at about 7 tons per square foot., or slightly under 3.8 tons per square foot in excess of natural soil pressure. This second requirement will be met by leaving the dredge wells of the caissons filled only with water from the level of the tremie base seals to the tops of the caissons proper at El. —35. The bottoms of the caissons are to be supported on tremie concrete seals 19 to 21 ft. thick, which transmit the soil pressures into the several bearing walls of the caissons, the walls themselves being heavily reinforced to resist direct bearing pressures and arch action that may occur against the exterior walls.

Timber cofferdams extending from the

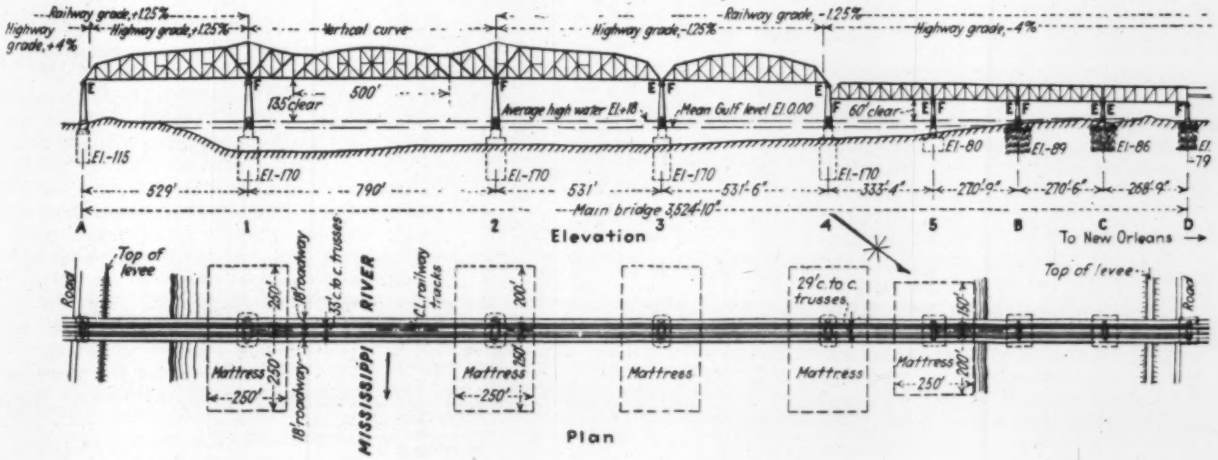


Fig. 1—The main river crossing of the Mississippi River bridge at New Orleans will consist of a through cantilever structure. With approaches, the bridge will be more than 4 miles long. Willow mattresses have been sunk over each pier site to eliminate scour during caisson-sinking.

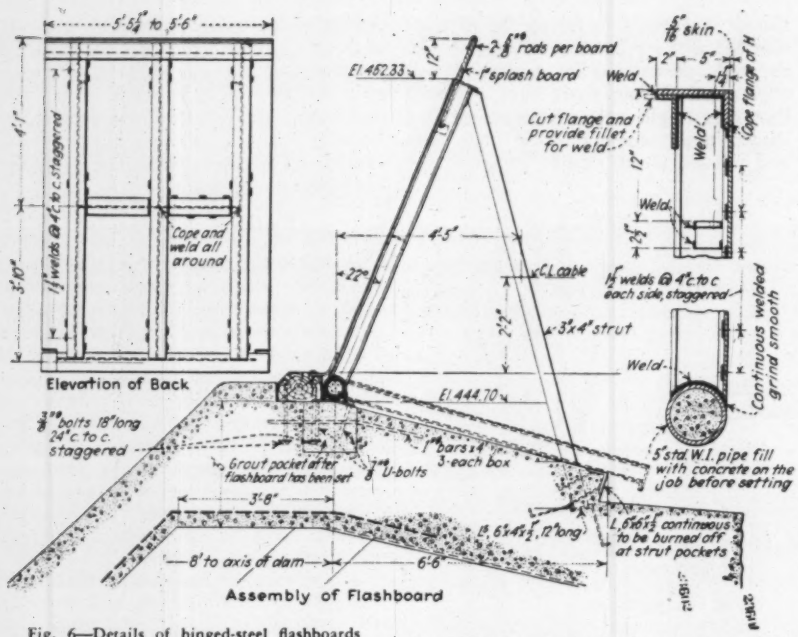

Elevation of Back
Assembly of Flashboard
Cope and weld all around
1½" welds @ 4" c. to c. staggered
2-⅞" rods per board
1" splash board
El. 452.33
Weld
Cut flange and provide fillet for weld
⅛" skin
Cope flange of H
4'-5"
C.L. cable
1½" welds @ 4" c. to c. each side, staggered
3"x4" strut
Continuous welded grind smooth
El. 444.70
⅞" bolts 18" long 24" c. to c. staggered
Grout pocket after flashboard has been set
1" bars x 4 3 each box
⅞" U-bolts
5" std. W.I. pipe fill with concrete on the job before setting
3'-8"
8' to axis of dam
6-6
15 6"x4½"x½" 12' long
L 6"x6"x½" continuous to be burned off at strut pockets

Fig. 6—Details of hinged-steel flashboards for raising the spillway crest 8 ft.

nent flashboard-operating cableways over the spillway structure. The operations of raising and lowering the flashboards and of placing the struts are carried on from a man-car suspended from a second cableway a short distance downstream from the main operating cableway. This second method of operating the boards will undoubtedly be used more frequently than the first method.

After completion of the dam a test of these unusually large flashboards was made under full 8-ft. head, three panels being lowered in 6 min. and raised against full head in 9 min. At this rate the 48 sections could be lowered under flood conditions in about 1½ hours, and under similar conditions could be raised in less than 2½ hours.

Construction methods

The original construction in 1924 exhausted the near-by sources of sand and gravel for concrete aggregates, so that it was necessary to utilize a shallow river deposit 2 miles below the dam. The construction work incidental to raising the height of the dam extended from the summer of 1930 over into 1931, with a winter shutdown of about 3½ months required by the Public Service Commission of New Hampshire. As this gravel pit was submerged during the high water of spring and early summer, it was necessary to haul practically the entire requirements of sand and gravel during the fall of 1930 and store the run-of-bank materials in large stockpiles at the site of the work.

Power shovels were used for digging and loading the material at the pit. A small speeder shovel reclaimed it at the stockpiles, loading into small trucks that dumped the unwashed raw materials direct into the receiving hopper for the bucket elevator at the principal storage bin. Oversize material was passed into a crusher whose output fed back into the main bucket conveyor. Everything was elevated to the top of the main bin where the material was washed and classified. A large semi-portable steel-frame secondary bin equipped with measuring hoppers with a 1-yd. mixer below is fed from the larger timber-storage bunkers with bucket elevators, making an installation that required very few men for economical operation.

The principal equipment consisted of cableways. A large electric cableway with 2½-in. main cable on 80-ft. timber A-frames was placed directly over the axis of the dam. A secondary cableway with a 1¼-in. cable on 50-ft. A-frames was installed 25 ft. farther downstream, thus fully covering the reconstruction work. Concrete was placed by means of 1-yd. bottom-dump buckets handled by the cableways direct from the mixer to the forms with no chuting or rehandling. For the long downstream buttress extensions these buckets were transferred to car frames on sectional track placed directly on the top extensions of large sturdy form panels, thus avoiding all manipulation of the concrete in the forms except the usual spading or rodding and eliminating any tendency to separation of aggregates.

The large form panels as well as all reinforcing steel were placed with the cableways, and much of the excavation was also handled with this equipment.

A stiff-leg derrick was set up at each abutment to handle the muck from the corewall excavation as well as from the extensions to the dam abutments themselves, and a caterpillar crane served as general utility equipment.

The project was constructed for the Public Service Company of New Hampshire, of which F. H. Mason is chief engineer and A. R. Schiller is general manager. The initial and final construction of the dam and power-house were both designed and built by the Ambursen Construction Co., of New York. John L. Frazier was superintendent in charge of construction, and H. D. Resseguie served as resident engineer representing the owner.

Handling Smoke Abatement

PROGRESS in reducing city smoke nuisance and damage is being accomplished largely by extensive use of educational methods, according to discussions at the American Society of Mechanical Engineers' Chicago convention, June 26-30. The discussion indicated that numerous technical problems still await solution.

William G. Christy, smoke abatement engineer for Hudson County, N. J., reported success in dealing with the smoke from shipping in New York harbor. Education in proper combustion and information as to the economic losses due to smoke are better means of solving the problem than punishment for the production of smoke, he stated. Makers of boilers and furnaces still pay too little attention to the matter of how their product is to be operated and what fuel is to be used. For example, equipment designed for anthracite is being sold for use with bituminous coal.

Thorough attention needs to be given to the human element, since the best equipment will produce smoke if not properly operated, and the fireman's care depends in part on the attitude of his superiors. While smoke abatement essentially is an engineering problem, political interference often hampers enforcement, and there is the difficulty of making the public and plant managers "smoke-conscious."

These ideas were reflected in the discussion, but it was pointed out that in many cases smoke campaigns are short-lived, due either to direct opposition or to incompetent political management. In Chicago success is being attained through educating firemen in the proper handling of boilers and informing plant managers as to its economic value, according to Frank Chambers, city smoke inspector. Legal action is reserved for dealing with direct opposition.

Methods of measuring cinders (from stokers) and fly-ash (from pulverized coal) were outlined by A. C. Stern, of Stevens Institute of Technology. This study was started because of agitation for legal limits of permissible emission of cinders and fly-ash. Sulphur compounds in waste gases are also a source of trouble, however, as was pointed out by H. F. Johnson, of the University of Illinois. Methods for removing these compounds from the flue gases are available.

Foundations Get Under Way on New Orleans Bridge

Special open caissons, to be sunk 180 ft. through 90 ft. of water by the sand-island method, have closely spaced wells in the concrete walls that provide access to the cutting edges —Willow mattresses sunk at each pier to prevent scour

THE foundation contractor on the combined railway and highway bridge over the Mississippi River, 10 miles above the "old city" section of New Orleans, has initiated his work by weaving and sinking large willow mattresses over each of the river pier locations. These mattresses, four of which are now finished, are required to prevent scour as the caissons are being sunk. That such scour is a distinct possibility was revealed during the taking of test cores, when holes 12 to 14 ft. deep occurred around the 4-in. drill casings. Under normal conditions the river current has a velocity of 4 miles per hour. With the mat work completed, caisson work will be started. The deepest of these foundations will be sunk 180 ft. below mean gulf level, or about 195 ft. below the maximum working stage. It is planned to do this work without resorting to compressed air, as four of the caissons will be sunk far below limits for pneumatic work, and unusual open-bottom caissons have been designed for the purpose.

The New Orleans bridge is being built jointly by the Public Belt Railroad Commission and the state of Louisiana at an approximate cost of $13,000,000, loaned by the Reconstruction Finance Corporation. Upon completion the bridge will be used by the Southern Pacific Railroad in addition to the Public Belt Railroad and the main state highway into New Orleans from the northwest. The bridge and approaches have a total length of 23,000 ft., the river crossing being a cantilever structure with a 790-ft. main span and 530-ft. anchor arms. The remainder of the main bridge superstructure consists of a 530-ft. through span and four deck-truss spans varying in length from 333 to 269 ft. This span arrangement was made necessary to meet navigation requirements. The long viaducts will have tower spans of 45 ft. and intermediate spans varying in length from about 69 to 82 ft.

A vertical clearance of 135 ft. above mean high water is provided for the main bridge structure. The approach structures will be constructed with the railroad decks on descending grades of 1.25 per cent. The vehicle roadway, however, will descend on a 4 per cent grade. There are two of these vehicle roadways, each 18 ft. wide, supported outside the through trusses on cantilever extensions of the floor beams. On the approaches the roadways are supported on brackets cantilevered from the vertical members of the deck-truss spans and from the columns of the viaduct towers.

Foundation design

Nine piers will support the main bridge, two on pile foundations on the river side of the east levee, which is a "making" bank, one on piles behind the east levee, one caisson pier in the comparatively shallow water near the east bank, four caissons in the deeper portion of the river and one caisson behind the levee on the west bank. Any future shifting of the river channel would probably be toward the west, which is a "cutting" bank, and this accounts for the use of a caisson instead of a pile foundation behind the west levee.

The caissons planned for the bridge are of unusual design and will be open-dredged to the unprecedented depth of 180 ft. below gulf level. Until recently the deepest caisson was at Hawkesbury, Australia, open dredged to a total depth of 162 ft. below water level. During June of this year a caisson for the Atchafalaya River bridge at Morgan City, La., was sunk 176.5 ft., and one of the Trans-Bay Bridge piers at San Francisco will go down 226 ft.

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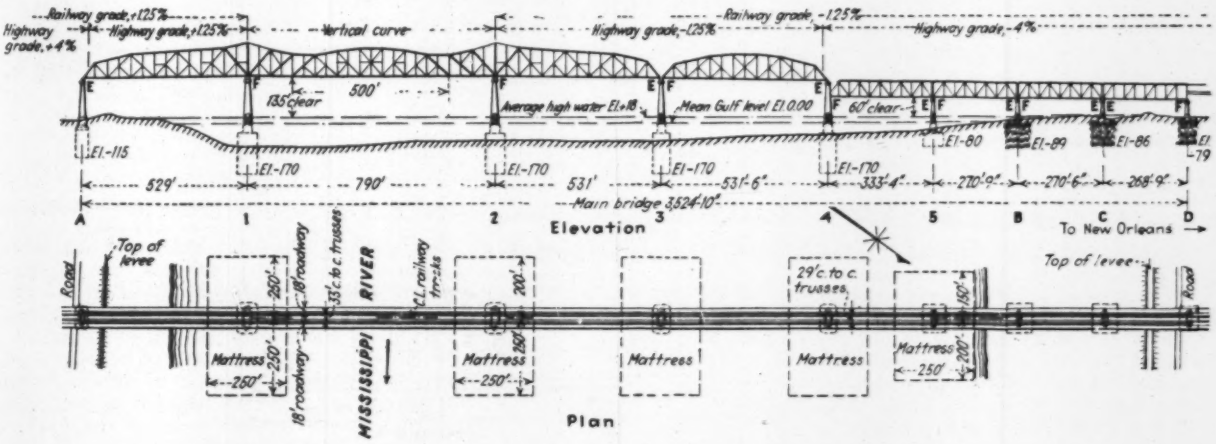


Fig. 1—The main river crossing of the Mississippi River bridge at New Orleans will consist of a through cantilever structure. With approaches, the bridge will be more than 4 miles long. Willow mattresses have been sunk over each pier site to eliminate scour during caisson-sinking.

main caissons at El. —35 to El. +15 will permit unwatering to El. —35, so that the distributing blocks and pier shafts may be constructed in the dry. The dredge wells that will not be covered by the concrete distributing blocks will be provided with heavy timber covers securely locked in place to prevent them becoming filled with silt and thereby increasing the weight on the soil.

An unusual feature of the caisson design is the provision, in all exterior and interior concrete walls, for vertical tubes, closely spaced, which will provide access to the cutting edges for jetting, drilling, or cutting away obstructions encountered during sinking. Soil borings made previous to the design, as well as borings made recently by the foundation contractor at the sites of the several piers, indicate sunken logs occurring at various depths from the river bottom down to El. —200. This, with the fact that certain of the clay strata are unusually tough, dictated the use of the vertical tubes in the caisson walls.

The original design contemplated floating caissons that would be sunk to the river bottom, then cut through the mattress and open-dredged to the required depths. The foundation contractor, however, adopted the sand-island method. A continuous steel-plate shell, of a diameter large enough to surround the caisson with several feet of clearance at the corners, will be sunk onto the previously placed willow mattress. The shell will then cut through the mattress and be allowed to sink several feet into the soft material of the river bottom. The circular section of mattress within the area of the shell will be removed, and the inclosed area will be filled with sand approximately to water level. The reinforced-concrete caisson will then be constructed in the dry on top of the sand fill, and when a suitable height of concrete is placed and sufficiently cured the dredging operations will be started. The caisson will be alternately dredged and built upward until it reaches the required depth. After the cutting edge of each caisson has penetrated some distance below the river bottom, it is planned to remove the sand island and re-use it at the site of another pier.

This method of pier construction is similar to that used for the piers of the Suisun Bay Bridge of the Southern Pacific Railroad Co., which were constructed by this same contractor (*ENR*, Jan. 30, 1930, p. 174). Certain features of the sand-island method have been patented by M. F. Clements, of St. Paul, Minn., who is consulting engineer for the contractor.

Mattress-sinking

The work on the main bridge foundations, due to the velocity of the current, the depth of the river and the unstable nature of the riverbed encountered, promises to be difficult and exacting. Before the caissons can be sunk, woven willow mattresses must be placed to prevent

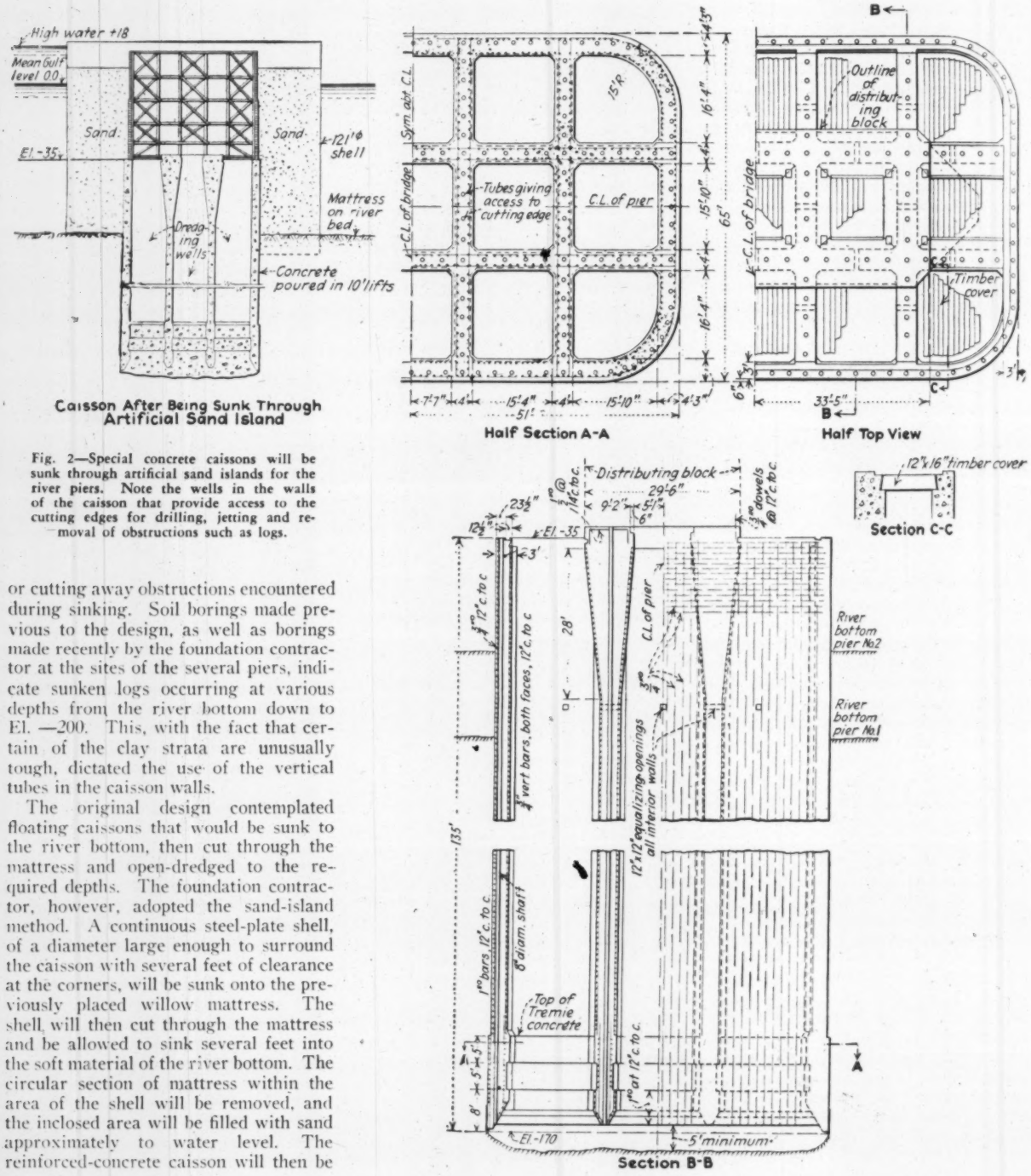


Fig. 2—Special concrete caissons will be sunk through artificial sand islands for the river piers. Note the wells in the walls of the caisson that provide access to the cutting edges for drilling, jetting and removal of obstructions such as logs.

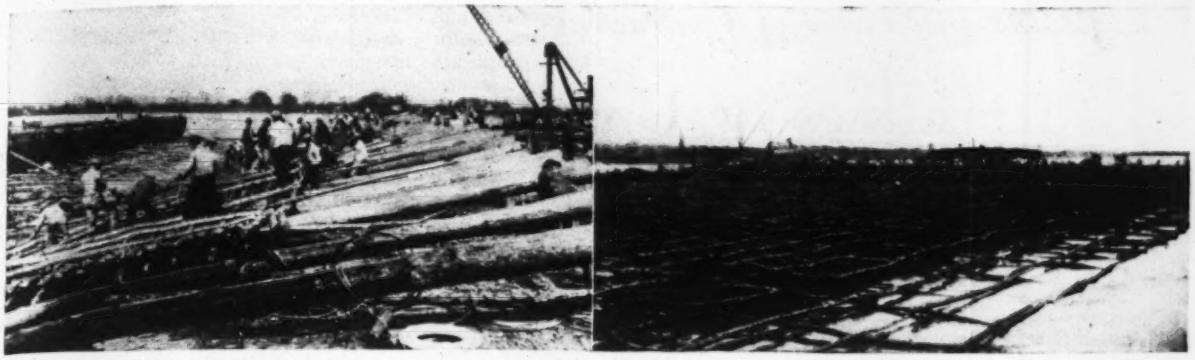


Fig. 3—The weaving of a willow mattress on inclined ways mounted on a barge is shown in the left view. As the mat is completed it is allowed to slide into the water, as shown at the right. The mats are sunk by weighting with stone, beginning at the upstream end.

scour. These mats for the four piers in the deeper portion of the riverbed, where the full effect of the current is felt, are 250x450 ft. in plan, the longest dimension being up- and downstream. The mattress for the eastern shore pier, which is less exposed, is 250x350 ft. in plan.

Mat-weaving has followed practice that has been well standardized in flood-control work on the Mississippi River. The weaving barge is equipped with inclined ways on which the mats are assembled from willow branches transferred from a supply barge. As the weaving progresses the barges are moved downstream and the mats are allowed to float on the surface. Fig. 3, left, shows the start of the mattress weaving; a section 20 ft. long and the full width of the mat (250 ft.) has been completed and is being wired up preparatory to launching. Fig. 3, right, shows the same mattress almost completed, with the weaving ways in the background.

After completion, the mattresses are uniformly ballasted with sandstone and lowered from the barges by manila lines connecting the upstream headers of the mats to timber heads set about 10 ft. apart on the barges. The upstream end is lowered to the bottom first so that the current will assist rather than hinder the work. After the mats are on the river bottom, additional ballast is dropped from barges moving slowly over the mat area. Experiments were carried out using captive weights dropped from the barges, to see how far the current would carry them downstream. The stone barges are kept above this point so that all ballast will land on the mats.

Three permanent anchors are placed upstream 600 ft. from the edge of each mattress. Steel cables from these anchors run through the entire length of the mattress and are attached to the downstream headers. Buoys attached to the four corners of the mats mark their location.

Four triangulation stations, two on either side of the river, are used in determining the accurate location of all mats and piers. Attendants at these stations, by means of visual signals and whistles, notify the tug captains and engineers on the barges when they are in proper location for the performance of their particular tasks.

Modjeski, Masters & Chase are designing and supervising engineers for the entire work, and Moran & Proctor are their consultants on the foundations. Siems-Helmers, Inc., St. Paul, has the $3,083,185 contract for the main substructure, and Bilhorn, Bowers & Peters, St. Louis, are subcontractors on the mattress work. MacDonald Engineering Co. is contractor on the approach substructure. The American Bridge Co. has the $2,618,670 contract for the main superstructure, and McClintic-Marshall Corp. has the $3,226,789 contract for the approach superstructure. C. Glennon Melville is engineer in charge of construction.

Length of Pavement Required for a Car to Pass a Truck of Maximum Length

By J. N. Arnold
Instructor, Engineering Drawing, Purdue University, Lafayette, Ind.

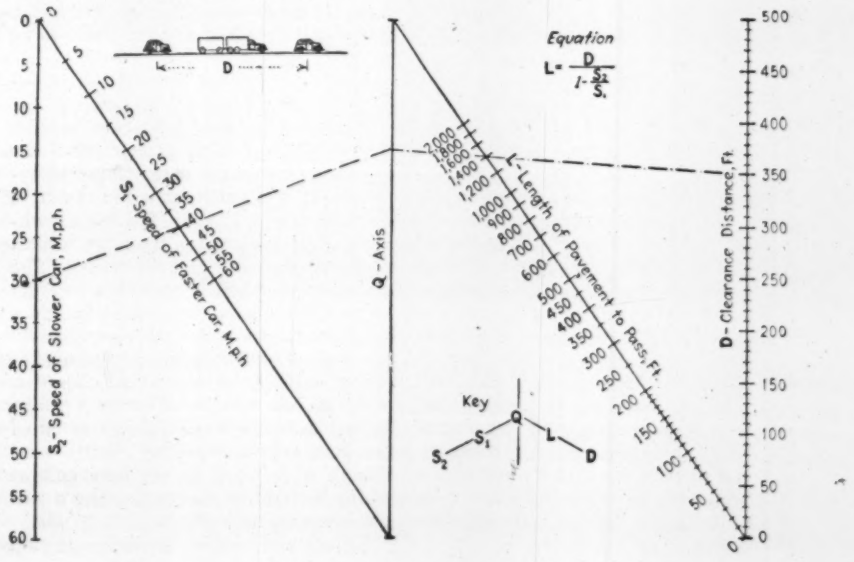
BECAUSE of recent legislation in a number of states limiting the overall length of trucks and trailers permitted on the highways, the length of clear pavement required for cars to pass trucks of maximum length is important in determining locations of highway markers at curves and on hills where the road is visible for only a limited distance.

The accompanying chart may be used to solve for the length of clear pavement required when the speeds of both the car and the truck are known. An example illustrates its use:

S_2 = speed of truck = 30 miles per hour.
S_1 = speed of car = 40 miles per hour.

The length of pavement required to gain 350 ft. (150 ft. behind to 150 ft. ahead of a 50-ft. truck) is found by drawing a straight line from 30 on the S_2 scale through 40 on the S_1 scale to the Q axis. Through the point thus located draw a straight line across the L scale to 350 on the D scale and read L = 1,400 ft.

Chart for computing length of pavement required to pass a truck.


Equation
L = D / (1 - S2/S1)
S2 - Speed of Slower Car, M.p.h
S1 - Speed of Faster Car, M.p.h
Q - Axis
L - Length of Pavement to Pass, Ft.
D - Clearance Distance, Ft.
Key
S2 S1 Q L D

Ten Business Problems of Contracting

III—Ethics of Unbalanced Bidding

Unbalanced bids may often prove advantageous both to the owner and the contractor—Justification depends on objective of the act—Typical examples of sound and unsound objectives reviewed

By J. L. Harrison
Senior Highway Engineer, Bureau of Public Roads, Washington, D. C.

PREVIOUS ARTICLES have indicated that securing business is the most important aspect of contracting, and that securing this business at the right price is an essential closely associated with this major objective. But the art of bidding goes even a little farther than this and touches the matter of protecting the contractor against losses of certain kinds that are of rather frequent occurence in construction work.

The customs and practices that govern in the construction field result in bids or tenders of one sort or another. Such tenders may be in a lump sum only or, and rather more customarily, it may be required that they be itemized. In the latter event the total of the cost of the various items is the amount of the bid and the basis on which the award is made. Progress payments are also based on the prices assigned to the various listed items. Both the possibility of unbalancing bids and any advantage to be derived from so doing are results of this practice.

In asking for itemized tenders it is presumed that each item will be so priced that it will carry its fair share of the total cost of the job. If it does, the bid is a balanced bid. But quite often contractors find it desirable to raise the price on some items and correspondingly to reduce the price on others; when this is done the result is an unbalanced bid.

Protection of working capital

Various purposes underlie the submission of unbalanced bids. Of these one of the most common is a desire to protect working capital. The expense that is involved in securing business and in bringing supplies, materials of construction, and equipment onto the site of the work is likely to be considerable; but as expenditures for these purposes do not, in and of themselves, produce much of anything that is of tangible value to the owner, it is quite common to phrase contracts so that under a rigidly balanced bid preliminary items of expense will be liquidated only as the work progresses. This is not unfair to the contractor, and it has at least the appearance of being of advantage to the owner. However, it does tie up working capital, which, in turn, reduces the rate of turnover on whatever working capital the contractor may have. This is a matter of some importance to him, for in the construction field turnover has much the same relation to profit that it has in the mercantile field.

This matter will be treated more at length in a subsequent article; but in view of the tendency to draw specifications in such a way that preliminary expenses are liquidated only as the construction progresses, it may not be inappropriate to observe here that the amount of working capital that is required in order to finance construction evidently depends on the promptness with which the contractor's claims are met and the frequency with which they are presented. If, as an illustration, the payment for a given job is to be made as a lump sum when the work is completed, the contractor or those who furnish the materials and supplies must provide working capital for the whole undertaking.

But, if instead of a single payment at the completion of the job, the owner liquidates all expenses every fifteen days, the contractor needs only a relatively limited amount of working capital. Under an assumption that six months will be required in which to execute this work, it is evident that more than ten times as much working capital is required under the first type of contract as is required under the second type. The contractor may not furnish all of the working capital when he is working on a single-payment contract; but if he does not furnish it, someone else must; and if someone else does, the cost to him in the way of discounts lost on the materials used or in the way of interest on funds borrowed will be considerable.

Under the second type of contract—that is, under a contract calling for prompt and frequent payments and for the liquidation of all expense that has been incurred to the dates of these payments—it is possible for a contractor to discount all bills if he himself has a working capital no greater than the largest single estimate he expects to receive. The discounts so taken are apt not to seem large when viewed separately, but as a lump sum they provide rather an astonishing rate of return on the capital used—indeed, so large a return that operation under the second type of contract can be highly profitable from the standpoint of the contractor's working capital even though it is taken at the bare cost at which it could be executed under a single-payment contract.

Unbalanced bidding for the protection of working capital has this situation primarily in mind. It is, then, merely an effort to force the owner to liquidate preliminary expenses somewhat more promptly than the terms of the proposal and the specifications propose, so that the money spent on them can be recovered and put to work rather than allowed to lie idle.

To effectuate this purpose, the amount of such expenses as those named and of other similar expenses is included in some relatively large item or items that will be completed soon after the work starts—as, for example, in the price named on the excavation and the foundation for a building, no allowance on account of these expenses being made in other items.

Assuming that the owner is dealing with a reliable contractor, a matter to which his attention is properly directed before the award is made, his position as an owner is improved by countenancing this sort of unbalanced bidding, for it improves the contractor's financial position. For the same reason it improves the position of the bonding company. As the interests of all parties to the contract are forwarded by unbalanced bidding of this particular type, it is to be encouraged as a legitimate business practice.

But, while unbalancing bids for the purpose of collecting preliminary expenses as promptly as possible appears to be a sound business practice, and therefore is to be encouraged, some of the extensions of this practice, though having about the same objective, are not so definitely sound and ought usually to be avoided. These extensions of the idea of balancing bids to protect working capital usually fall in one of two general classes: those that seek to avoid the effect of the retent on working capital and those that seek to bolster up an inadequate working capital.

Escape of retained percentage

The custom of retaining a percentage of each estimate until the job has been completed is common throughout the construction field. From the financial standpoint this practice is unqualifiedly bad, for it tends considerably and quite unnecessarily to increase the amount of working capital required. Nevertheless, the practice persists, and in order somewhat to avoid its effect contractors sometimes load important items to be completed during the earlier part of the job. To attain this objective, the load-

ing need not cover the full amount of the retent, as some profit is normally expected. If, as an illustration, the retent is 10 per cent and the expected profit is 5 per cent, by loading items to be completed early in the job with 5 per cent of the job cost the working capital can be supported during the earlier stages of the work. Naturally it will be depleted rather rapidly toward the end of the job, but this depletion would occur in any event, and the fact that the period involved is reduced may be of considerable assistance to the contractor.

Unbalancing a bid for this purpose may be quite definitely in the interest of the contractor and sound business practice from his standpoint, but as it tends toward overpayment during the earlier stages of the job the owner can hardly be expected always to look on it with favor. Evidently this practice is of most advantage to contractors who have suffered some depletion of working capital or who have taken more work than the working capital at their disposal would entirely justify, since it tends to reduce the amount that it is necessary to borrow and so to reduce interest charges. In spite of this advantage to the contractor, this type of bidding is not to be recommended, because of the tendency it involves to require the prepayment of a portion of the job expense. But it cannot be said that there is anything ethically unwholesome about it, or that when a bid of this type is presented by a thoroughly reliable contractor it should necessarily be rejected, for the owner may, in practice, find it desirable to render such a contractor the financial assistance that a bid of this sort implies.

Bolstering lack of capital

The second extension of the practice of so bidding as to protect working capital is open to more serious question because it is more definitely an effort to obtain financial assistance from the owner. In bids of this type the prices named on the items that will be completed first are deliberately raised by such an amount that working capital will be built up as these are completed. In the nature of the case, the price offered on items completed toward the close of the job are correspondingly reduced. The effect is a frank prepayment on account of the items handled toward the close of the job by whatever amount the items scheduled for early completion are loaded. Again, there is nothing ethically unsound in a bid of this sort, and the arrangement of payments which such a bid contemplates may be of very definite assistance to the contractor.

From the standpoint of the owner the principal question is whether he desires to assist the contractor financially to any such extent as bids of this type imply. As a rule, he does not; and if he does not, such bids are naturally and properly rejected. It may be observed that the tendency to reject bids of this sort is encouraged—and properly so—by the fact that they are seldom entered except when a contractor is very short of working capital, a fact that tends to make their acceptance dangerous. Both owners and bonding companies are warranted in viewing bids as badly unbalanced, as bids of this kind usually must be to effectuate the purpose that underlies them and as involving an undesirable risk.

Capitalizing classification

In the general field of excavation, unbalanced bidding is common. Excavation is widely classified under three headings—common, loose rock and solid rock. Sometimes a two-way classification—common and solid rock—is used. There are, of course, other classifications. While recently a good deal of earthwork has been awarded on an unclassified basis—that is, without any allowance for classification—a far larger amount is done under specifications that provide for some form of classification. In order to establish a basis for comparing bids, the amount of material falling in each of the established classifications that field examinations, surveys, etc., have indicated must be handled is customarily shown. Again, the presumption is that each item will be correctly priced. Actually, several considerations tend to encourage unbalanced bidding. Of these, the problem of recovering the cost of bringing in equipment and of removing it, of depreciation while it is on the job and of related costs will be the first considered.

Each of whatever classifications there may be on the job under consideration will be responsible for a certain amount of the probable sum total of these expenses which, in practice, originate in the owner's statement of the work that there is to do. Nor will any change in the quantities actually encountered much modify these costs; for as a rule the job must be supplied with whatever equipment the owner's preliminary classification indicates will be required in executing the work, and ordinarily it must be retained on the job until the work is completed. In practice, then, these costs are relatively fixed.

On the other hand, while as a general rule the preliminary surveys are so accurate that the total quantity of material to be handled on jobs involving excavation of any sort is pretty accurately stated, the amounts assigned to the various classifications are subject to a good deal of variation. This is particularly true of jobs involving relatively small amounts of rock and, on bridge work, involving small amounts of wet excavation. Incidentally, it may be remarked that while perhaps more commonly encountered in the field of earthwork, possible changes in the amount of minor items offer about the same problem in many other construction fields.

Obviously, if all the costs mentioned above and any similar costs not specifically noted that will be generated by some specific item, as solid rock, are included in the price bid on solid rock, and later the quantity of solid rock is considerably reduced, there will be a proportionate part of these costs that cannot be recovered. For this reason it always is good practice to study rather carefully the reliability of any classification, of the amounts assigned to all minor items for that matter, to determine which item or items will be subject to the least change, and then to transfer all of the relatively fixed charges such as those outlined above to the items on which the least change is to be expected.

Here the evident and the entirely legitimate purpose in unbalancing the bid is to provide for the recovery of general items of expense that will be incurred on account of certain possibly variable construction items that will not be altered by even rather considerable changes in the amount of this work performed under these variable items. If this is done skilfully and accurately, the contractor will benefit appreciably in the event the variable items are modified, while the owner pays no more than it is reasonable to presume he would have paid had quantities been stated correctly in his proposal. The bonding company's position is improved because the contractor's position has been measurably protected. In brief, then, this sort of unbalanced bidding is entirely legitimate and is to be encouraged because it results from the use of good business judgment in the avoidance of a risk rather commonly encountered in handling earthwork and sometimes encountered in other construction fields.

Where there is a disposition to suspect the honesty or the intelligence with which classification will be handled in preparing estimates on completed work, contractors occasionally bid a uniform average price for all classifications. Bids unbalanced in this way rest on an assumption that the quantities given in the proposal are accurate, or at least that there is less danger that they will prove inaccurate than that the classification made when the work is performed will be inaccurate. No injustice to the owner is intended in unbalancing bids in this way, but a dangerous risk is unnecessarily assumed. In an extreme case that came to the writer's personal attention, a contractor who took some 225,000 cu.yd. of excavation on this basis—supposed to be roughly 160,000 cu.yd. of common and 65,000 cu.yd. of rock—found on opening up the work that the net result would be about 40,000 cu.yd. of common and 125,000 cu.yd. of rock. The loss suffered was terrific.

The owner is hardly in a position to question the fairness of bids of this sort, for they are based on confidence in the preliminary estimates made by his engineering staff; but bonding companies should scrutinize such bids with care on account of the fact that in spite of supposedly careful technical investigation serious errors in preliminary esti-

mates of classification do occur. For the same reason this type of bidding ought never to be attempted by a contractor unless he is very sure that the preliminary classification is accurate. And, by the same token, both owners and bonding companies are justified in opposing bids of this type. The accuracy with which preliminary estimates of classification are prepared simply does not justify them.

But, over and above this fact, a serious problem is presented in paying for the work performed. If the more expensive classifications are taken out first, working capital is rapidly tied up. If the cheaper classifications are handled first, the contractor can be seriously overpaid during the earlier stages of the job. Extreme cases are not the rule, but the owner is likely to encounter difficulties on this score which it is better to avoid.

Speculating on error

The fact that there may be considerable error in the technical investigations on which preliminary estimates of earthwork and its probable classification are made provides the basis for a certain amount of speculative bidding. To illustrate, if as the result of private investigation it seems reasonable to assume that the amount of one of a number of classifications, such as loose rock, will be considerably greater than the owner's estimate indicates, the bid may be unbalanced by decreasing the price asked on other items and correspondingly raising the price on loose rock. If the owner's estimate proves correct, the contractor's recovery for the job as a whole is not affected; but if his private estimate proves correct, the loading given to this particular item results in considerable extra profit. The owner loses correspondingly.

The practice of unbalancing bids in this way is frankly objectionable. Ethically it is beyond defense, for clearly the purpose is to obtain an excessive price at the expense of the owner—a price which the owner had no intention of paying. From the practical standpoint also, unbalancing bids in this way is unsound, for it places the contractor in the position of speculating on his own work. At best, this is dangerous, for though the objective is profit, there is no certainty that the result may not be a serious loss. Both the contractor's estimate and the owner's estimate may be in error, or—and of more importance in its effect on the risk that bidding in this way involves—the fact that prices have been unbalanced to take advantage of the owner may lead the owner, in self-defense, to readjust the work so as to avoid the effect of the loaded bid and to take advantage of the correspondingly reduced prices offered on other classifications. The risk that bonding companies assume in covering bids that have been unbalanced in this way is quite as serious as the risk that contractors assume in entering such bids. All such bids should be rejected as unethical and wholly contrary to the owner's interests.

But, while bids that are unbalanced for the purpose of taking advantage of the owner may properly be rejected, the mere fact that a bid is unbalanced apparently for this purpose does not, of itself, justify rejection; for, in spite of appearances, the actual objective may be quite reasonable. The plain truth is that a bid may be rather badly unbalanced merely for the purpose of preventing some course of action that the contractor does not wish to have followed and to which he has good reason to object. The contractor, for instance, may have reason to believe that a bridge foundation is likely to be set a few feet lower than as shown on the plans. To discourage this change and to protect himself in the event it is made, the price of wet excavation may be loaded. Where plans for excavation show a minor amount of work, it is not uncommon to find that the price named for this work has been loaded, the purpose again being to prevent any change in the plans that will considerably increase the amount of this sort of work, or to force the negotiation of a price on it if the plans must be changed.

Bids of this sort are perfectly proper, for their objective is protection rather than price advantage. They do the owner no damage, for the price named on the thing he says he desires is not disturbed. They assist the contractor and protect the bonding company because they create a situation in which the owner must negotiate a reasonable price if important changes are made.

Bids are sometimes unbalanced in order to obtain an advantage in bidding and an advantage in price at no disadvantage to the owner. Special knowledge of the conditions that will govern during the construction stage of the work affords the basis for such bids. A case that recently came to the attention of the writer offers an excellent illustration. Near a Western project, some 18 miles of gravel surfacing on a rural highway, three gravel pits had been located in such position that the maximum haul was about 3½ miles and the average haul about 2 miles. These pits were shown in the plans for the job and formed the basis for most of the bids entered. A price was asked on two items: (1) furnishing, loading, dumping and compacting the gravel, including the first half-mile of haul; and (2) extra haul per half-mile or fraction thereof. The lowest bid, based on the use of the three known pits was: item 1, 40c.; item 2, 10c., which yielded an average price for the job of 55c. per cubic yard. One contractor bid: item 1, 47c.; item 2, 5c., which yielded an average price of 54½c. per cubic yard for the job as a whole.

When this work was started it developed that the low bidder had discovered two additional gravel pits so located that the average haul for the job as a whole could be reduced to about 1¼ miles. Under the assumption that the bid of 10c. on the hauling was correct, the information the contractor had secured through private field investigations had enabled him to underbid others and still to improve his bid on item 1 by nearly 18 per cent. As all parties were benefited by the low bidder's offer, the fact that his bid was unbalanced could not reasonably be held to be improper, particularly as this was the only means open to him to use his information without making it public.

Establishing false prices

No discussion of unbalanced bidding would be even measurably complete without at least a brief reference to the practice of unbalancing bids in order to develop a basis from which to negotiate with subcontractors. This matter requires no amplification here, for it will be perfectly obvious that if prices are unbalanced with this end in view, the purpose is to drive down the prices paid. In the long run it is just as important that the subcontractors obtain business at a fair price as it is that the main contractor obtain it at a fair price. Building up a series of false prices as a basis from which to conduct negotiations with business associates savors of sharp practice. It is not a line of procedure that can easily be associated with contracting as a business based on integrity and responsibility.

Water Resources of the Walla Walla River

The Walla Walla River, which drains a portion of the Blue Mountains in northeastern Oregon and southeastern Washington, has power and storage possibilities whose development may readily conform with and promote the further agricultural and industrial development in this region. A report just released by the Interior Department describes four power sites and two reservoir sites on this river. The combined capacity of the power sites is about 20,000 hp., and of the reservoirs about 36,000 acre-ft.

The report, which was prepared by the Geological Survey in connection with the classification of lands valuable for power and storage, is accompanied by maps and was prepared by Arthur Johnson, hydraulic engineer. It also contains geologic descriptions of the dam sites and territory adjacent to the river by C. E. Erdmann, geologist. The report is open to public inspection at the main office of the Geological Survey in Washington, D. C., and at the district office of the Survey at the Federal Buildings in Portland and Tacoma. It includes 61 pages of text and numerous tables, diagrams and photographs. The maps, including the maps of the dam sites and the river profile, are published on four sheets and can be purchased from the Geological Survey at Washington, D. C., for 40c. a set.

Gypsum-Incased Steel Columns Subjected to Fire Tests

New tests demonstrate importance of some form of positive bond to hold gypsum blocks in place, also the increased efficiency obtained by plastering the blocks

By Nolan D. Mitchell
Structural Engineer, U. S. Bureau of Standards, Washington, D. C.

FIRE TESTS of six gypsum-covered steel columns have recently been made at the Bureau of Standards, Washington, D. C., that supply new information on methods of positive anchorage of the gypsum block and on the effect of plaster in decreasing shrinkage and increasing the fire resistance of the protected columns. The tests supplement tests of five columns included in the series conducted jointly by the Associated Factory Mutual Fire Insurance Companies, the National Board of Fire Underwriters, and the Bureau of Standards at Underwriters' Laboratories, Chicago, during the years 1917-'18 and published as Technologic Paper No. 184 of the Bureau of Standards, 1921. (Also *ENR*, July 21 and July 28, 1921, pp. 106 and 146.) In these early tests the covering failed due to checking, shrinking and disintegration of the blocks, causing them to loosen and fall off, generally before the temperature of the steel exceeded 212 deg F.

In the recent series the steel columns were of H-section fabricated from four angles 3x2½x½ in., and one plate 6x½ in. The effective length was 10 ft. 4 in., and the slenderness ratio, l/r, 91.2 One end was fixed and the other was mounted on a spherical bearing. One column, No. 1, Fig. 1, was protected with 2 in. of poured monolithic gypsum concrete containing 12½ per cent of wood shavings by weight of dry materials, and finished with ½ in. of 1:3 sanded gypsum plaster. The other five columns had protective coverings made of blocks cut from gypsum partition tiles and constructed as shown in the details of columns Nos. 2 to 6, Fig. 1. In one of the five block coverings, No. 3, the re-entrant spaces of the column shaft were filled with block and mortar. In the others these spaces were not filled. This covering also had ties made of strips of wire lath laid in the horizontal mortar joints to bond the blocks, the same as was used in three of the earlier tests. On the other four coverings channel-shaped cramps made of No. 12 gage band iron were pushed down into holes in adjacent blocks in each corner of each course. The cramps were inserted into the core holes of the hollow blocks, or into holes drilled into the solid blocks.

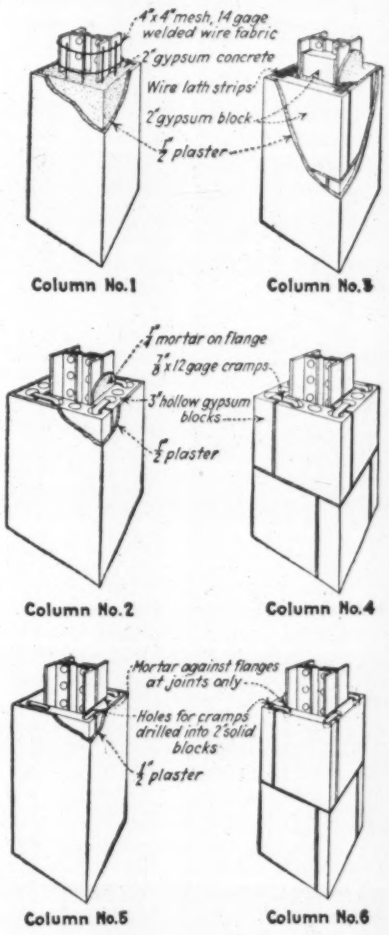


Fig. 1—Details of columns covered with gypsum block, plaster and poured gypsum, which were subjected to fire tests.

The tests were made by setting the columns centrally in the furnace and subjecting them to a gas fire until they failed under applied loads somewhat in excess of loads computed by the American Institute of Steel Construction column formula. The excess for columns Nos. 1 and 2 was 14 per cent, and for the others 4 per cent of the computed load. The fires were controlled to give exposing temperatures in accordance with the Tentative American Standard Specification for Fire Tests of Building Construction and Materials. Base-metal thermocouples were used to measure temperature at twelve locations in the furnace and at ten points on the column shafts.

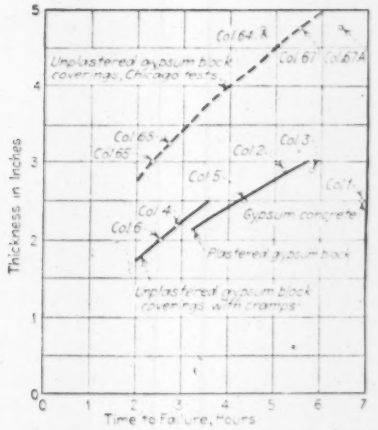


Fig. 2—Relation of thickness of gypsum covering to time of failure of columns in fire tests.

About two-thirds of the exposed plaster fell from the covering of column No. 1, mostly during the first 1½ hours of the test, but none fell from those with block coverings. All the plastered coverings remained in place until the columns failed. On the other hand, the unplastered block coverings, notwithstanding the cramps to bind the blocks, began to fall off before the columns failed, thus hastening failure to some extent. It was observed, however, that when the blocks began to fall in the recent tests the temperature of the steel was 3 to 4½ times as high as it was in the Chicago tests when falling of the blocks began, thus indicating better performance for the cramps than for corrugated wall ties or wire lath strips laid in the horizontal mortar joints.

The use of sanded gypsum plaster on the gypsum-block coverings gave a greater increase in fire resistance than did the use of the cramps. Column No. 3 failed after 5 hr. 47 min. of exposure compared with 2 hr. 21½ min. for column No. 65 of the previous series. The coverings of both columns were of approximately the same total thickness outside the steel and were alike in construction except that ½ in. of the thickness of the covering of column No. 3 was sanded gypsum plaster with the usual white finish. The results of the tests of the two pairs of columns, Nos. 2 and 4 and Nos. 5 and 6, are even more informative as to the effectiveness of gypsum plaster when applied over the block coverings. The coverings of each pair were alike except that one was plastered and the other was not, and that No. 2 had mortar between the blocks and the column flanges. The times to failure in the four tests were as follows: No. 2 (plastered), 5 hr. 10 min.; No. 4, 2 hr. 52 min.; No. 5 (plastered), 4 hr. 21 min.; and No. 6, 2 hr. 33 min. The 138-min. difference in fire resistance of the first pair can be attributed to the plaster finish, and the

Abstract of a complete report appearing in the June *Journal of Research* of the Bureau of Standards.

FIRE RESISTANCE OF STEEL COLUMNS WITH GYPSUM COVERINGS

Covering						Min. Net Area of Column and Covering, Sq. In.	Ultimate Fire-Resistance Period, Hours
Kind of Covering	Thickness Outside of Column Flange, In.	Metal Bond		Internal Construction	Surface[1] Finish		
		Kind	Placement				
None	0[4]	None	.....	Bare column	None	13	¼
2-in. solid block	2	Cramps	In corners[2]	Air space surrounding steel. No fill	None	85	2½
2-in. solid block	2½	Cramps	In corners[2]	Air space surrounding steel. No fill	Plaster	105	4½
2-in. solid block	3	Corrugated wall ties	In joints[3]	Solid fill of gypsum block and mortar	None	130	2½
2-in. solid block	3	Strips of wire lath	In joints[3]	Solid fill of gypsum block and mortar	Plaster	150	5½
2-in. solid block	3½	Strips of wire lath	In joints[3]	Solid fill of mortar and broken gypsum block	None	130	2½
4-in. solid block	4½	Corrugated wall ties	In joints[3]	Filled with hollow block and mortar	None	240	4½
4-in. solid block	4½	Strips of wire lath	In joints[3]	Solid fill of mortar and broken gypsum block	None	240	6
3-in. hollow block	3[4]	Cramps	In corners[2]	Air space surrounding steel. No fill	None	95	2½
3-in. hollow block	3½[4]	Cramps	In corners[2]	½ in. of mortar on flange. No fill	Plaster	120	5
2-in. fibered gypsumconcrete	2½	4x4-in. mesh wire fabric	Around steel column	Monolithic poured covering	Plaster	130	7

[1]Plaster consisted of ¾ in. of 1:3 sanded gypsum plaster with white finish.
[2]Pushed down into holes in tops of blocks. See Fig. 2.
[3]Wall ties or strips of wire lath laid in horizontal mortar joints.
[4]Total thickness, including core space in blocks. The net thicknesses were 2.2 and 2.9 in., respectively.

¼ in. of mortar on the column flange. The difference in resistance of the second pair, 108 min., must be attributed to the plaster alone. The plaster finish added to the fire resistance in greater proportion than the ratio of the squares of the resulting net thicknesses of covering outside the steel.

The relation of net thickness of covering to time to failure in the tests of plastered covering without fill conforms closely to the formula,

$$t = 35(d^2 + 1)$$

in which t is the time to failure in minutes, and d the average net thickness in inches outside the steel. Columns with unplastered block coverings, with cramps to bind the blocks, failed at 85 per cent of the time computed by the formula, and those with wire lath or corrugated wall ties in the horizontal joints (Chicago tests) failed at 40 per cent of the time so computed.

The fire resistance of the column with monolithic-fibered gypsum-concrete covering was approximately double that given by the formula for those with plastered block coverings. The principal differences between the covering of this column and those with solidly filled block coverings were its greater proportion of wood shavings, monolithic construction and a well-distributed reinforcement, but the individual effect of these and other factors on the fire resistance is not known.

Fire-resistance ratings for gypsum-protected columns derived from the previous and recent series of tests are given in the table. A study of the table will show advantages to be derived from the use of cramps or plaster with block coverings and that the reinforced monolithic covering is much more effective than the block.

H. J. Schweim, chief engineer of the Gypsum Association, arranged for the tests, and S. H. Ingberg, chief of the fire-resistance section of the Bureau of Standards, planned and supervised them. The author was in direct charge of the work.

Lock Construction at Dam 3, Tennessee River

Excavation and wall-concreting are progressing rapidly with simple plant set-up—Lock being built to permit possible increase in height of dam

CONSTRUCTION of the lock that is a part of the proposed Dam No. 3, a navigation and power project on the Tennessee River (*ENR*, May 11, 1933, p. 579) 15 miles upstream from the Wilson Dam (Muscle Shoals), has proceeded continuously since it was started last January under the direction of the Corps of Engineers of the United States Army in accordance with a contract entered into with the federal government by Stevens Brothers & The Miller-Hutchinson Co., New Orleans.

When the decision was made to provide a 9-ft. stage in the Tennessee from the Ohio to Knoxville, Tenn., a distance of 648 miles, the Corps of Engineers selected the site where work on the lock is now under way. At that time a 15-ft. lift dam was contemplated. Later the requirements were changed, and the height for which the contract was let provided for a dam having a 39-ft. lift.

When the Tennessee Valley Authority was created early this year, it was thought advisable to modify the design of the lock to permit a still higher dam to be built at this site, in case such procedure should appear desirable in the future. Work is going ahead rapidly on the lock, with all features of the latter being built to permit the increased height to be provided if desired. No final plans have been prepared for the dam and power house, the construction of which, it is now expected, will be undertaken by the Tennessee Valley Authority.

Standards adopted for the 9-ft. navigation stage on the Tennessee are being followed in the construction of the lock at the Dam No. 3 site. The lock chamber is to be 60 by 360 ft. in plan. The pool above the Wilson Dam provides slack water to this project, and the latter will make 9-ft. navigation possible about 51 miles farther upstream, assuming that a 39-ft. dam is built at this point.

Limestone underlies the entire locality in which this project is situated. Ledge rock at the site of the lock was covered with comparatively little detritus. The contractor has unwatered the site with a braced sheetpile type of cofferdam, both ends of which abut against the sheer rock face that forms the shore line of the stream. The river bottom is of a character that practically eliminates seepage into the coffer.

Excavation for the wall footings was carried down an average of about 4 ft. below the rock surface to obtain foundations that were satisfactory under the

Fig. 1—Line-drilling and broaching solid-rock foundation for lock walls. Shallow excavation in stratified river bottom uncovers uniform ledge of limestone which forms the foundation of the lock and the proposed dam.

maximum heads to be expected on the structure. The sides of the excavation were line-drilled or channeled, and blasting was done in such manner as to avoid damage to the foundation. With a total head of 12 ft. at the bottom of the excavation and a river stage of 8 ft. on the outside of the cofferdam, no seepage has been found at any point. The rock also is unusually tough and dense for this type of material.

Spoil from the excavation was removed by crawler cranes and shovels loading into dump-body trucks for delivery to a waste bank upstream from the cofferdam. Excavation and removal of the rock spoil and concrete-pouring were carried along simultaneously without any interference. This was largely made possible by the installation of a traveling gantry that spanned the space between the lock walls. A whirler mounted on this gantry handles excavated rock, concrete and other materials. The propelling mechanism of the gantry, as well as whirler operation, is controlled by the crane operator.

Concrete is mixed in a central plant erected on the offshore side of the cofferdam. Three 1-yd. tilting mixers in this plant are mounted under bins and batchers, to which sand and gravel are delivered from barges on the river side by a floating derrick. Cement from Cape Girardeau on the Mississippi River in Missouri is delivered in bulk to the project in tows of two barges, each holding eight cars. The delivery route is down the Mississippi from Cape Girardeau to the Ohio, up the latter to the Tennessee, and thence on that river to the site, a distance of about 350 miles. At the work the cement is pumped from the barge alongside the cofferdam to the batchers. Although the maximum distance of delivery is about 250 ft. and the lift is as much as 35 ft., no difficulty is experienced in moving the bulk cement. Only three men are required in the barge to watch the pump and do some cleaning up.

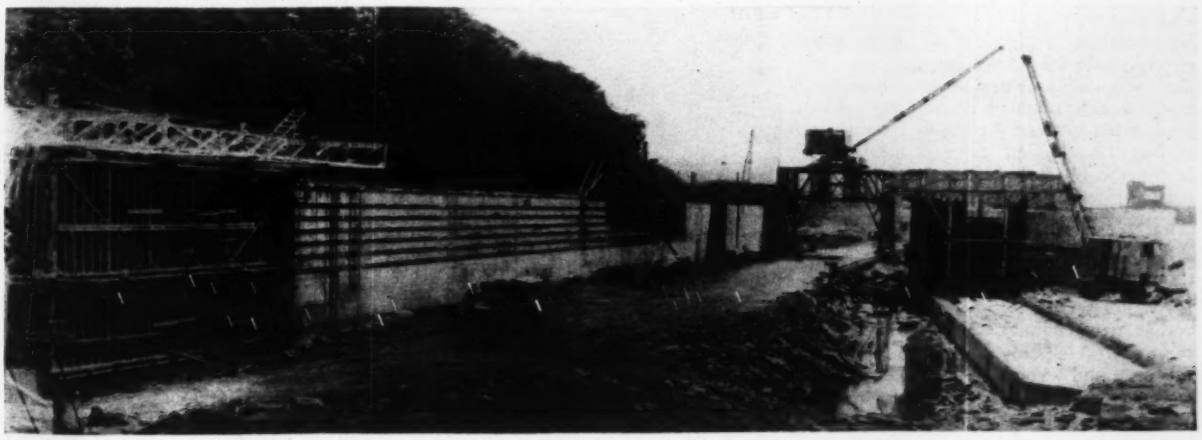
Motor trucks handling bottom-dump buckets deliver concrete from the mixing plant to within range of the crawler cranes that are used to reach the forms. As a general practice the traveling gantry crane operating along the central runway is used in pair with one or the other of the cranes on the outside of both walls. Concrete is usually poured in only one wall at a time, the outside crane on the other wall then being used to handle spoil from the excavation.

Each section of movable wall form consists essentially of a structural-steel frame, 30 ft. long and of a height to suit the lifts in which the concrete is poured. This structural-steel frame is mounted on a pair of long legs at one end and on a short pair at the other end. Each pair of legs carries double-flanged wheels running on a wide-gage track. Wooden side forms lined with Prestwood are swung from this traveling steel frame.

With a form section spotted at the point desired, the end legs are blocked in place so that they cannot move lengthwise of the wall. The side forms are then brought to line and grade by adjustments on the steel frame. Heavy cross-bolts between the forms, which are removed after the concrete is poured, are then used to anchor the side forms accurately in the determined position. When the concrete has hardened sufficiently, taps on the cross-bolts between the side forms of a section are loosened, these bolts are pulled, the end blocking of the legs is removed, and the section is shifted forward to the next set-up. The shifting is done easily by means of a fall line handled by one of the cranes.

The walls are being built in three lifts of about 15 ft. each. As soon as one lift is poured the length of the wall, the forms are jacked up to the next lift. The pair of long legs of the section of forms then travels on a track on top of the first lift. For the first pour at the new height

Fig. 2—Whirler on traveling gantry, spanning space between the lock walls, permits unobstructed roadway for trucks handling rock spoil and concrete.



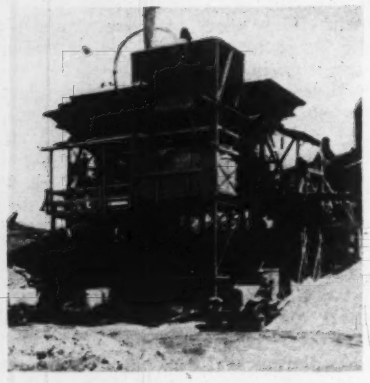


Fig. 3—Central mixing plant against outside leg of coffer is served by floating derrick in the river and delivers to bottom-dump buckets in which concrete is hauled to point of operations by trucks.

the short legs are shored up on the top of the first lift of concrete. After that they travel on the concrete of the second lift.

Funds for this project were provided under the Emergency Construction Act for the relief of the unemployed by means of river and harbor works. Under the contract forces may be worked on 6-hour shifts only. Most of the operations are conducted on a 24-hour basis. Since about 65 men are required to a shift, the contractor would have had to provide facilities for over 350 employees if a construction camp had been provided at the site of the lock. The desire also to furnish work for resident local labor to the fullest possible extent was a factor in deciding to house men in their own homes.

This combination of conditions has been met effectively by using motor buses to transport the entire force employed. These buses run between the job and the city of Florence, Ala., a distance of about 20 miles. They are operated by the contractor without expense to the men.

On account of one shift coming off as the next goes on, the buses run full in both directions. Stops to pick up and discharge employees are made going and coming at any place along the route. This scheme has worked well for all concerned.

The improvement of the Tennessee River has been under the direction of Major-General Lytle Brown, chief of engineers, with Major R. R. Neyland as district engineer in general charge, Captain H. D. W. Riley as military assistant in direct charge of construction, and G. P. Fleetwood as resident engineer.

Blasting a Cutoff Channel on the Saco River

Single dynamite blast cuts off ox-bow bend that threatens bridge on state highway between Fryeburg and East Fryeburg, Maine

By L. D. Barrows
Chief Engineer, State Highway Commission, Augusta, Me.

THE SACO RIVER has been for some time a matter of concern to the Maine state highway engineers at an ox-bow bend in the channel northeast of Mt. Tom. Parallel to the Saco and a short distance from it (Fig. 1) was another stream over which the Osgood trestle had been constructed. If the Saco, during a period of high water, made a new channel across the intervening land, thereby turning its waters under the Osgood trestle instead of under the Walker bridge as formerly, serious consequences to bridges and highways might occur. To prevent such an occurrence, a survey of the bend was made in June, 1932, and the decision was made to construct a new channel across its neck.

The new channel was located at the most desirable point as regards soil conditions and volume of excavation. Scrapers were employed at first to remove the soil, but the presence of roots, stumps and submerged logs made the work so difficult and expensive that other methods were sought. Dynamiting the new channel was proposed, and after consultation with an explosives expert it was decided to employ that method to clear the channel.

Fig. 2—Appearance of channel opened by blast of 1,500 lb. of dynamite. Flow through channel has now increased the volume of earth removed to more than 7,000 cu.yd.

Holes were made with a post-hole digger at intervals of 5 ft. along the new channel, and to a depth of 7 ft. Special care was taken to bore the holes to uniform depth and spacing. In each hole 35 to 40 lb. of ditching dynamite was placed, the heavier loading being used where stumps or logs were encountered. The entire line of charges was exploded simultaneously, as there was sufficient moisture in the soil to propagate the shock wave from charge to charge. The explosion produced a cut (Fig. 2) 35 ft. wide at the top, 12 ft. wide at the bottom and 8 ft. deep.

Because of the consistency of the soil no efforts were made to make the new channel as wide as the river to be diverted. The action of explosive charges, when loaded as described, loosens about twice the amount of material actually excavated. A survey of the blasted trench by the engineers convinced them that when the water entered the new channel it would be enlarged quickly and adequately to accommodate the river flow even during flood periods. There was an 8-in. fall along the new channel. The blast required approximately 1,500 lb. of dynamite to excavate about 1,400 cu.yd. of soil containing stumps and submerged logs.

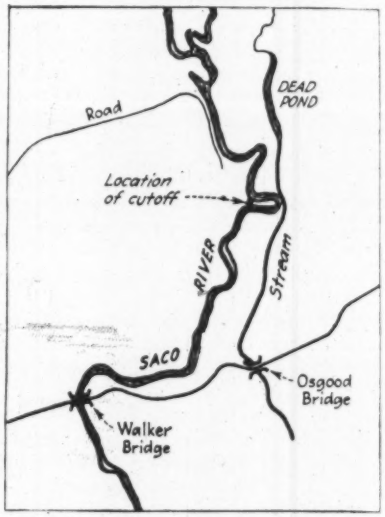


Fig. 1—Route of Saco River through Fryeburg township, Maine, showing bend cut off by blasting channel across the neck.

On May 18, 1933, an inspection of the blasted cutoff by state highway engineers disclosed a channel 60 ft. wide, and soundings indicated a depth of 18 to 20 ft. It was noted that silt was being deposited at the old channel entrance, thus making a natural and permanent barrier to the flow of the Saco River toward the near-by stream. It is estimated that the action of the explosives and the force of the volume of water passing through the new channel have removed 7,000 cu.yd., or 4¼ cu.yd. per pound of explosive employed.

Chart Determines Hauling Costs and Number of Trucks Required

By R. D. Gladding
Warren Technical Service, Inc., Boston, Mass.

THE COST of hauling by truck consists of two main items. The first includes expenses that depend upon the actual distance the truck travels, such as wear and tear, fuel, tires, etc.; the second is fixed expense, such as interest, depreciation, taxes, insurance, overhead, etc. The second item can be reduced to the cost per hour, to which is added the wages of the driver, and the sum called the hourly fixed charge. Both of these costs may be closely determined for any particular type of truck.

In order to estimate properly the hauling cost on road construction, it is necessary to determine in advance the average length of haul and the time consumed in making one round trip at this distance. The calculation of the average haul requires no explanation. The time elapsed in one round trip is composed of (1) the time waiting while loading and unloading and the average time lost by delays; and (2) the time in transit loaded and empty. Time lost will vary between wide limits; for example, an asphalt distributor may require as much as 30 min. to load and unload, whereas only 3 min. may be needed when hauling bulk materials from bins. Time in transit will vary with the nature of the roadbed and the type of truck. Over new grade with hills the average speed, loaded and empty, might be 10 miles per hour, while the same truck might average 25 to 30 miles per hour on a level paved highway. These values must be assumed for the particular problem.

Let g = the running cost per mile. K = hourly fixed charge. L = length of haul in miles. t = lost time per trip in minutes. S = average speed in miles per hour. C = cost per trip in dollars.

Then $$C = 2gL + \left(\frac{2L}{S} + \frac{t}{60}\right)K \qquad (1)$$

The item $2gL$ is the running cost per trip. The formula also serves to show that $\frac{2L}{S}$ parts of an hour are consumed in running time and $\frac{t}{60}$ parts of an hour in lost time, both at K dollars per hour.

For example, assume a truck with a fixed charge of $1.20 per hour and a running charge of $0.09 per mile. Further consider that the haul is over new level grade and that 15 miles per hour can be expected and a loading and unloading time of 6 min. Substituting these values,

$$C = 2 \times 0.09L + \frac{2 \times 1.20L}{15} + \frac{6 \times 1.20}{60}$$
$$= .18L + .16L + .12 = .34L + .12$$

It is more usual to consider the cost on the basis of a rental value per day with the running charge included. That is, g in formula (1) becomes zero and

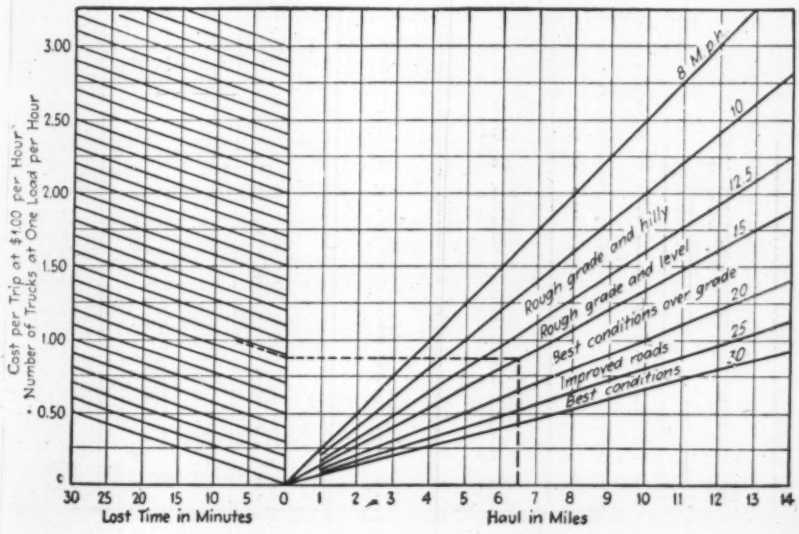
$$C = \left(\frac{2L}{S} + \frac{t}{60}\right)K \qquad (2)$$

Assume the truck rent to be $24 per day of ten hours, or $2.40 per hour, $S = 15$ and $t = 6$. Substituting,

$$C = .32L + .24$$

These two examples are chosen because they illustrate the approximate nature of formula (2). The value of K in (2) was selected as equal to the total cost per hour of example (1) at $L = 6$ miles.

Diagram for determining cost of haul in construction work.



If we consider K in formula (2) to be $1, we can construct a chart to cover all usual conditions provided the result is multiplied by the actual hourly charge K. This has been done in the accompanying diagram.

For example, assume K = $2 per hour, $S = 15$, $t = 6$ and $L = 6.5$. This problem is illustrated on the chart and gives C a value of $0.96 for K = $1. The cost of the trip when K = $2 is $1.92. If the truck hauls 5 tons the cost per ton is $.38 and the cost per ton-mile is $.38 ÷ 6.5 = $0.0585.

It is valuable to be able to estimate the number of trucks that will be needed at different stages of construction as the haul varies in length. The same chart can be used to solve this problem.

In formula (2) the value of $\frac{2L}{S} + \frac{t}{60}$ is the actual time in hours consumed in making one round trip or in delivering one load. Call this time T. Then the number of trips a truck would make in one hour would be $\frac{1}{T}$. If Q = the number of loads to be delivered each hour, then the number of trucks required would be Q divided by $\frac{1}{T}$ or QT. Substituting for T its value $\frac{2L}{S} + \frac{t}{60}$ the number of trucks required would be,

$$N = Q\left(\frac{2L}{S} + \frac{t}{60}\right) \qquad (3)$$

This formula is identical with (2) if we consider K and $Q = 1$. For example, assume that it is desired to know how many trucks will be required on a concrete slab job when 35 batches per hour are to be hauled, $t = 6$, $S = 15$ when the mixer is 6.5 miles from the batching plant. If the trucks carry two batches, then 17.5 loads per hour are required. This is the same condition as imposed before as to length of haul, etc., and the chart gives a value of 0.96 for $Q = 1$. For $Q = 17.5$ loads per hour, the number of trucks, $N = 17.5 \times 0.96 = 16.8$, or 17 trucks.

It is evident that in nearly every case the results obtained by any theoretical method will have to be increased because the number of trucks required will not come out even. Therefore the costs should be increased to take care of this.

Assume that the number of trucks needed is determined for each mile of advance on a concrete paving job using one-batch trucks, 35 batches per hour, $S = 15$, $t = 6$.

LENGTH OF HAUL	1	2	3	4	5	6	14
N (chart)...	8.4	12.6	17.5	21.7	26.2	31.5	68.3
N (actual)..	9	13	18.0	22	27	32	69
% Increase.	7	3	3	1	3	1.5	1

That is, for short hauls from 5 to 10 per cent should be added to the cost derived, and for longer hauls only 5 per cent or less plus the expense of extra trucks considered necessary to insure adequate supplies at all times.

Survey of Traffic Volumes Through Building Exits

Flow of actual traffic in normal and congested periods determined by field observations in office buildings, stores, theaters and railway stations

By H. B. Houghton and J. H. Courtney
Associate Engineers, Bureau of Standards, Washington, D. C.

ONE OF THE first questions that arises in the planning of a building is how many exits are necessary in order to assure the occupants reasonable safety in case of fire. If there are too few, disaster and loss of life may occur; if there are too many, valuable space is taken from the rentable area of the building without contributing a proportional amount of safety. Just what constitutes a safe means of egress is a question on which opinions differ widely.

In its consideration of minimum requirements for means of egress, a joint conference composed of representatives from the building-code committee of the U.S. Department of Commerce and from the building-exits-code committee of the American Standards Association (the latter including the membership of the committee on safety to life of the National Fire Protection Association) felt that a study of the movement of people through exits of different types, would assist materially in arriving at rational conclusions. Such problems as the relationship between rate of discharge and size of exits, the ratio of the rate of discharge up and down stairways to that on the level and through doorways, the capacity of stairways when discharging crowds of people, and factors affecting flow seemed to warrant thorough investigation.

Procedure

As a preliminary step in the solution of these questions; the available data on traffic studies through exits were obtained and reviewed. For the most part, these data were rather meager. The tests themselves were found to vary over a wide range. The many variables influencing flow through exits, including the proportion of treads and risers, story height, width and type of landing, introduced many complications. Also the familiarity of the occupants with the exits, the physical make-up of the occupants, the probability of fatigue in discharge, especially from high buildings, and the necessity for hurried exit had a direct bearing on the interpretation of test results. Opinions could reasonably differ widely on the same set of data. It is not always possible to predict how people will act under fire conditions. Yet some general average on the rates of flow likely to be expected was necessary if requirements were to be formulated that would provide adequate safety for the occupants of the building and, at the same time, assist in effecting practical and economical design.

Field survey

To check the information at hand on rates of flow through exits and to find out if all the basic facts had been fully obtained or developed, a field survey was instituted. A number of the larger government office buildings in Washington, D. C., were studied, as well as department stores and theaters. Fire drills in some of the temporary government office buildings were also observed.

Some difficulty was encountered in obtaining congested conditions. The survey was therefore extended to New York City, where checks on the movement of crowds were made in the Grand Central Terminal, at some of the subway and elevated exits, in one of the theaters, and at other points where there appeared to be a likelihood of the exits being used to capacity.

Methods used in checking flow

All observations were made with stop-watches and with mechanical hand counters. At the start, two observers with hand counters and another with a stop-watch took readings. After a few trials, which showed an error of about two or three people in every 300 when the counters were read every 30 sec., only one counter was used. It was also found that if the observer checking the time read the counter at the half-minute intervals, rather than asking for the reading from the man counting, who must keep a close watch on the flow, that there was much less chance of losing the count of a few people that would pass by while the man taking the count was making a reading.

In recording the discharge on stair-

TABLE I—SUMMARY OF OBSERVATIONS OF TRAFFIC FLOW ON STAIRWAYS

Location	Width of Stairs		Slope		Vertical Height		No. of Persons Discharged Per Ft. of Width Per Min.		Sq. Ft. Per Person	Travel	Remarks
			Rise	Tread			Average	Maximum			
	Ft.	In.	In.	In.	Ft.	In.					
U. S. Veterans Bureau	6	0	7	11	12	3	12	17	...	Down	
U. S. Veterans Bureau	6	0	7	11	12	3	12	16	...	Down	
U. S. Census (D Bldg.)	5	6	7	10½	..	..	11	17	...	Down	Exit during drill
U. S. Census (D Bldg.)	7	2½	6½	12	10	3½	11	18	...	Down	Exit during drill
U. S. Census (D Bldg.)	7	3½	7	12	10	3½	23	24	...	Down	Exit during drill
U. S. Census (D Bldg.)	4	0	7½	11½	20	7	13	14	...	Down	Exit during drill
44th St. Theater, N. Y. C.	6	3	7½	11½	15	0	11	13	8.1	Down	.....
44th St. Theater, N. Y. C.	6	3	7½	11½	15	0	10	12	...	Down	.....
Subway, 7th Ave. & 34	6	0	7	11½	8	9	17	18	...	Down	.....
G. C. T., Gate 115	5	0	7½	12	14	4	19	28	5.9	Down	.....
G. C. T., Gate 115	5	0	7½	12	14	4	21	25	5.2	Down	.....
G. C. T., Gate 115	5	0	7½	12	14	4	17	19	...	Down	.....
G. C. T., Gate 115	5	0	7½	12	14	4	18	21	4.8	Up	.....
G. C. T., Gate 115	5	0	7½	12	14	4	18	22	4.6	Up	.....
G. C. T., Gate 115	5	0	7½	12	14	4	16	18	...	Up	.....
G. C. T., Gate 115	5	0	7½	12	14	4	20	24	4.5	Up	.....
G. C. T., Gate 115	5	0	7½	12	14	4	16	19	5.6	Up	.....
G. C. T., Gate 116-117	5	0	7½	12	14	4	18	20	...	Up	.....
G. C. T., Gate 116-117	5	0	7½	12	14	4	18	20	...	Up	.....
G. C. T., Gate 116-117	5	0	7½	12	14	4	17	19	...	Up	.....
G. C. T., Gate 109-110	8	1	7½	11	18	9	12	16	4.9	Up	.....
G. C. T., Gate 109-110	8	1	7½	11	18	9	12	13	...	Up	.....
G. C. T., Gate 109-110	8	1	7½	11	18	9	13	16	6.6	Up	.....
G. C. T., Gate 109-110	8	1	7½	11	18	9	11	14	...	Up	.....
Civil Service Commission	4	1½	7½	11½	13	1½	17	18	...	Down	Exit during drill
Bureau of Standards	3	0	7	11½	13	5	31	34	4.1	Down	.....
Bureau of Standards	3	0	7	11½	13	5	29	29	4.1	Up	.....

G. C. T. = Grand Central Terminal, New York City.

TABLE II—SUMMARY OF STUDIES OF TRAFFIC FLOW ON RAMPS

Location	Width	Length	Slope	Area	No. of Persons Discharged Per Ft. Width Per Minute		Sq. Ft. Per Person	Travel	Remarks
					Average	Maximum			
G.C.T. Gate 111-112	6 ft. 4½ in	137 ft. 2 in.	1 in. 9.7	Sq. Ft. 875	14	21		Down	Commuters' traffic, all in one direction
G.C.T. Gate 111-112	6 ft. 4½ in.				22	22	7.5	Up	
G.C.T. Gate 111-112					17	19	9.5	Up	
G.C.T Gate 111-112					20	20	9.8	Up	
G.C.T. Gate 111-112					21	22	8.0	Up	
G.C.T. Gate 111-112					22	23	6.8	Up	
G.C.T Gate 109-110	6 ft. 4½ in.	137 ft. 2 in.	1 in. 9.7	875	20	22	8.3	Up	
G.C.T. Gate 109-110					19	19	7.3	Up	
G.C.T. Gate 109-110					19	19	7.6	Up	

ways or ramps two kinds of observations were made. Where rate only was checked, the counts were taken every 30 sec. during congestion, or the count was continued during the congested period and the time taken at the finish. Readings were not started until, as nearly as could be determined visually, the stairway or ramp was filled from top to bottom between hand-rails, and the count was stopped when this condition ceased.

Observations to determine both rate and capacity were made in two ways. In one case the observer with the stop-watch entered the stairway or ramp after it became filled, at the same time signaling to the observer with the counter to begin the count. The first observer then checked the time it took him to proceed up or down the stairway with the crowd, while the second observer checked the number of people passing during this time, thus enabling a computation to be made on the number of square feet occupied per person under those conditions of movement, tgether with a rate. In the other case, both observers stationed themselves at the same point. By picking out a certain individual as he entered the stairway, the time of his progress in the stairway was checked and the count taken of the persons passing in that time. This gave a check similar to that obtained with the preceding method. The second system offers the advantage of many more counts during the congested periods.

Counts on the rate through doorways were taken on either side of the doorway that permitted easy observation of the flow. It was found almost impossible to locate doorways used during congested periods where the doors were not fastened open for the convenience of the occupants of the building in passing out.

Flow on stairways

A large number of counts were taken on the flow both up and down stairways. Characteristics of this flow were noted. The average and maximum results of these readings are given in Table I. In many cases these average figures were obtained from 40 to 50 individual counts. It is recognized that these rates of discharge would be somewhat higher if they were calculated on the basis of 22-in. units, disregarding the effect of any dimensions less than half units.

As will be seen in Table I, the average results compare quite favorably with the maximum. There is an apparent tendency for a faster rate to obtain on the narrower stairways. At the Bureau of Standards, where a maximum rate of discharge downward of 34 persons per foot of width per minute was obtained, a group of people was collected at the top of the stairway and was asked to go down it as fast as possible without running. The rates of discharge were taken in both directions and checked. It was not found, as has been the case in some of the previous studies of this nature, that there was a differential in favor of a greater discharge *up* a single flight of stairs over that *down*. Under the same conditions the rate of discharge down was slightly higher than that up.

The low discharge rate for theater stairways is no doubt explained by the leisurely manner in which the patrons leave after the performance is over. A number of theater managers were questioned concerning the tendency for peopl to leave a theater by the same way in which they entered. Opinion on this was about equally divided. Some maintained that patrons use the same exit coming in and going out, while others expressed the belief that the nearest exit available at the close of the show is utilized.

Exit during fire drills was studied at some of the temporary government office buildings. The occupants of the building were never informed of the time when the drill would be staged. When the fire gong was sounded, the occupants left the building at random by exits that had been previously assigned to certain sections or floors. The one high rate of discharge for

TABLE III—SUMMARY OF OBSERVATIONS ON TRAFFIC FLOW THROUGH GATES AND DOORWAYS

Location	Width		No. of Persons Discharged Per Ft. of Width Per Min.		Remarks
			Average	Maximum	
Gates	Ft.	In.			
G. C. T. 113-114	5	0	24	26	
G. C. T. 111-112	5	0	26	26	
G. C. T. 109-110	5	0	29	29	
G. C. T. 107-108	5	0	30	32	
G. C. T. 107-104	5	0	30	33	
G. C. T. 41- 42	6	6	23	27	
U. S. Treas., single door	3	1	13	21	Door fastened open.
Nat'l Theater, pair doors	2	10	5	9	Doors fastened open. Not apparent capacity.
U. S. Dept. Com., pair doors	2	8	9	17	Doors fastened open.
Barnum & Bailey Circus	12	0	11	16	Passageway.
U. S. Census Bur.—C Bldg., pair doors	2	6	14	24	Doors fastened open. Taken during fire drill.
U. S. Census Bur.—C Bldg., pair doors	2	6	22	30	Doors fastened open. Taken during fire drill.
U. S. Census Bur.—D Bldg., pair doors	2	6	15	20	Doors fastened open. Taken during fire drill.
U. S. Census Bur., pair doors	2	6	13	25	Doors fastened open. Taken at 4.30 p.m.
U. S. Census Bur., C Bldg	3	0	15	20	Doors fastened open.
U. S. Census Bur., C Bldg., pair doors	2	6	17	18	Doors fastened open.
Nat'l Geographic Soc	2	8	22	26	
Nat'l, Geographic Soc	5	4	14	18	Doors fastened open.
Bur. Print. & Engraving, pair doors	2	9	18	27	Doors fastened open. Taken at 4.30 p.m.
Gov't Printing Office (revolving door collapsed)	Clear		22	33	Door collapsed.
U. S. Veterans Bureau, pair doors	3	0	12	18	Doors fastened open
Civil Serv. Commission, pair doors	2	6	20	21	Doors fastened open. Taken during fire drill.
Palais Royal, revolving	3	6	12	12	
Woodward & Lothrop, revolving	3	6	9	12	

these drills was obtained when one of the customary exits was closed off, thereby throwing twice the number of people on the exit under observation. The particular point studied was unusual in that it was at the foot of a half flight of a stairway measuring 7 ft. 3½ in., which connected at an intermediate landing with two stairways each 5 ft. 0 in., located on either side of the wider stairway (Table I).

For stairway widths from 36 in. up to 6 or 7 ft., having nearly the same treads and risers, the number of persons discharged is roughly proportional to the width of the stairs and could be represented by a straight-line curve. However, in the wider stairways there is a tendency for the proportionate discharge to diminish, especially if intermediate hand-rails are not provided. This condition is illustrated by the 8 ft. 1 in. stairway studied at the Grand Central Terminal, which had no intermediate hand-rail. Within a given stairway the rate of discharge remained constant; that is, when the movement was fast, the space between individuals was increased, and when the movement became slower as the density increased, the number of persons passing per foot of width per minute remained the same. This condition has been noted in other tests.

In all the stairways studied the appearance of the movement under random conditions of flow was similar. If the width was such as would accommodate three people abreast, when a rank of this number formed it was invariably followed by a rank of two, then one or two staggered until three again formed in line. The effect was to create a series of wedge-shaped units on the stairs occurring successively. It was also observed that as a crowd entered a stairway either for ascent or descent the movement on the first half of the flight was slower and therefore denser than it was on the second half. Conditions of maximum flow rarely occurred unless there was a crowd of people forming a sort of reservoir at the entrance to the exit.

From Table I it will be found that with random flow and ordinarily free movement on stairways about 5 sq.ft. of space is occupied per person. Referring to Table II, it will be seen that for ramps the average is about 8 sq.ft. per person. In the stairway studied at the Bureau of Standards, where an average of 4.1 sq.ft. per person was obtained with fast movement, only 1.4 sq.ft. of space per person was obtained by having the stairway filled with persons standing as close together as possible.

Flow on ramps

Studies of flow on ramps were made difficult because of the inability to find ramps of various sizes filled to capacity. A number of observations were taken in the Grand Central Terminal, and the results are tabulated in Table II. Attempts were made to check some of the wider ramps in the station, but they were always used so much below apparent capacity that counts would not have been worth while. From the table it will be seen that the rate of discharge for ramps is somewhat higher than that on stairways and somewhat lower than that on the level. The flow on the ramps showed the same characteristics as that already described for stairways.

Flow through doorways

A summary of studies of the discharge through doorways of varying widths is given in Table III. It should be noted in nearly all cases on the counts through doorways that the doors were fastened open while the readings were being taken.

The few counts made on revolving doors given in this table do not necessarily indicate maximum capacity, although they were taken over periods when the doors were being used as fast as appeared natural and convenient. The figures represent egress only. From observations of the action of people in revolving doors with various widths of leaves the case was found to be exceptional where two people attempted to go through the door together. It would seem, therefore, that the width in the case of a revolving door does not have any direct relationship to the number of persons discharged.

By comparison with Table I the rates of discharge for doorways will be seen to be somewhat higher than those for stairways. This supports a rather common theory that the ratio of the rate of flow on the level as compared with that on stairways is approximately as 4 is to 3. The number of persons discharged through doorways compares with that of stairways in that the number discharged is about proportional to the width. However, as in the case of stairways, there is a tendency for the discharge not to continue as a straight-line function when the widths become greater than 6 or 7 ft.

These present data were all on random conditions of flow. Together with the other data extent on this subject, the results have been utilized by the joint conference of members of the building-code committee of the Department of Commerce and members of the building-exits-code committee of the American Standards Association in the consideration of simple and practical methods for calculating the minimum required exit widths in buildings.

Shovel Places Concrete for Delaware Road

Commercial ready-mixed concrete discharged by agitator trucks into bottom-dump bucket substituted for dipper on power shovel

EXCEPTIONAL PROGRESS was made this spring in paving a section of three-lane concrete road in Delaware, using central-mixed concrete and placing it with a bottom-dump bucket substituted for the dipper on a power shovel. The new work is a part of the three-lane highway from Wilmington to Dover, which parallels the famous Coleman duPont Highway, the first 14-ft. width of which was built in 1915 and which has since been widened to four lanes. Together the new road and the old will form a 31-mile dual highway, separated by a parked strip.

In the specifications of the Delaware highway department, pavement concrete cannot be dumped directly or through chutes. The section of road being considered began about 2 miles south of Wilmington, where commercial ready-mixed concrete was available. It was 9,450 ft. long except one lane, which was 230 ft. longer. This made possible a maximum haul of 4 miles from the commercial mixing plant in Wilmington. For all this distance the duPont Highway was available. These conditions ensured quick delivery of ready-mixed concrete; and since the state specifications requiring a 1½-min. mix limited the capacity of a paving mixer to about 35 cu.yd. an hour, it was probable that the commercial ready-mixed concrete would give greater production. It was therefore decided by the contractor to use ready-mixed concrete delivered in agitator-body trucks. To meet the specifications prohibiting direct dumping and chuting, the contractor undertook an adaptation of available grading plant.

Ready-mixed concrete had been employed on two previous contracts in the vicinity of Wilmington by George & Lynch, of Dover, Del. These contractors had complied with the state highway specifications in one case by running the ready-mixed concrete through a paving mixer. On the second contract they distributed the concrete from the agitator trucks with a power shovel, using the standard dipper to carry and dump the concrete. The standard shovel dipper was not entirely satisfactory for this purpose. Drawing a lesson from this experience, the contractor for the work substituted a 1-yd. bottom-dump bucket to eliminate the spilling of con-

Fig. 1—Power shov l with bucket substituted for dipper receives concrete from agitator truck. Dumping ramp is pulled by shovel.

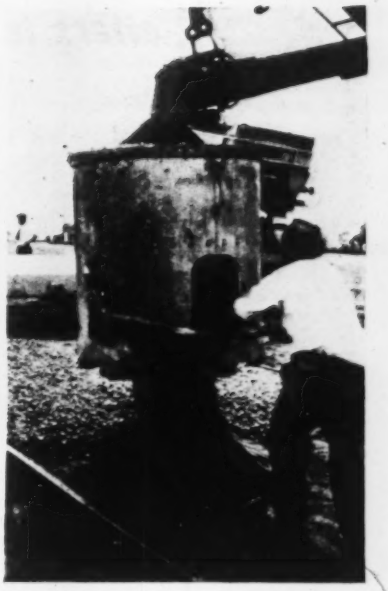


Fig. 2—Concrete bucket dumped by hand. Shovel operator aids spreading by employing crowding motion on dipper handle.

crete and the lack of dumping control experienced with the ordinary shovel dipper.

The height of the concrete bucket, as shown by Fig. 1, made it necessary to raise the rear end of the agitator trucks a couple of feet to dump into the bucket. This was accomplished by a timber ramp which the power shovel pulled behind it. This arrangement required all trucks to dump behind the shovel.

The specifications required that the three lanes be concreted independently. Paving operations for each lane were carried on from the south end to the north, with a maximum concrete haul to the south end of 4 miles. Six to eight trucks equipped with agitator bodies transport 4-yd. loads. At the job the power shovel and attached ramp operated along the level grade or grass strip at the proper distance from the forms to enable the shovel runner to distribute the concrete on the subgrade. Each truck backed up the timber ramp and discharged four times into the 1-yd. bottom-dump bucket. After the truck left the ramp, the shovel and ramp moved ahead about 16 ft.

A finishing machine, adjustable for use on both 10- and 11-ft. lanes, followed closely behind the concrete-placing crew. When the finishing machine was not in the way, the shovel made a quarter-turn from the agitator truck to the point of dumping the concrete. If, however, the finishing machine ran too close to the concrete-placing operation, the shovel had to rotate through a three-quarter turn in the opposite direction. One man on the subgrade (Fig. 2) opened the discharge gate of the bucket. By using the crowd on the dipper stick the shovel runner, who formerly operated a paving mixer, distributed the concrete fairly uniformly across the width of the subgrade between the forms. This uniform distribution lightened the work of the two shovelers who leveled the concrete for the finishing machine. A few small drippings fell from the truck discharge spout onto a steel plate, from which they were easily removed with a hand shovel and placed within the forms.

In addition to the usual finishing operations with straight-edges and long-handled floats behind the finishing machine, the surface of the concrete was raked with a fan-shaped aluminum broom. The ends of the metal ribs of this broom were bent down to form corrugations $\frac{1}{8}$ to $\frac{1}{2}$ in. deep in the mortar surface of the concrete slab. For five years Delaware has followed this practice of roughening the surface to provide better traction, with satisfactory results and with no resultant scaling of the concrete.

After curing for 24 hours with wet burlap, the concrete was covered with earth. Earth-curing was substituted in the state this year for calcium chloride, with the intention of providing more employment. Water was supplied by a pump through 2 miles of $2\frac{1}{2}$-in. pipe line. A smaller line would have been adequate for the needs of this contract, but the contractor of course was using equipment on hand.

Delaware has continued on this project the practice initiated last year of using cork expansion joints. Although the material is more expensive than premolded asphalt, the highway department feels that it has advantages in greater resiliency, probable longer life and freedom from surface humps. The rates of progress were as follows:

Average progress on the first lane (9,450 lin.-ft. of 8-in. slab 10 ft. wide) was 35.2 cu.yd. an hour. On the second lane (9,450 ft. long and 11 ft. wide) the contractor averaged 44 cu.yd. an hour. The third and longest lane (9,680 lin.-ft. of 10-ft. width) was completed at an average rate of 43.2 cu.yd. an hour. Production for the entire job averaged 39.7 cu.yd. an hour.

On the longest day's run, June 14, when 1,983 lin.-ft. of 10-ft. lane was completed in $9\frac{1}{4}$ hours, concreting progressed at an average hourly rate of 50.7 cu.yd. The work was started May 2 and was completed June 14. Concrete was placed on 24 days. Sundays and rain or wet subgrade accounted for sixteen days during the working period.

Engineers and contractors

Construction of the dual highway is being carried out under the general direction of W. W. Mack, chief engineer of the state highway department, with E. E. Downing, resident engineer, Wilmington, Del., in direct charge. H. E. Yates, general superintendent, supervised the work for the contractor, Vincent Schiavi, of Buffalo, N. Y.

Survey Control Planned for Los Angeles County

County surveyor and city engineering departments of Los Angeles, Calif., are planning to start this summer on a new system of survey control for Los Angeles County that is expected ultimately to be extended throughout the state. The system, authorized recently by the county board of supervisors and city officials, will establish new base lines and monuments from which future surveys can be made. Monuments will be established at about 2-mile intervals. The new system is to be run out under the direction of the joint committee on survey control through triangulation, which is to be headed by Oliver B. Keefe, office engineer and county surveyor's department. Work will begin from two base lines already established by the United States Coast and Geodetic Survey. One of the lines, said to be the most accurate measurement in the world, was made by the government for the late Albert A. Michelson when he was engaged in measuring the speed of light.

Letters to the Editor

Time of Mixing

Sir—I read with much interest the article on the time of mixing concrete by Major F. S. Besson (*ENR*, Feb. 9, 1933, p. 182).

Some time ago I had a series of tests made to satisfy myself about the time of mixing. On this job the concrete after leaving the mixer was dropped eight times and traveled through from ¼ to ½ mile of chutes.

I found that samples taken after the concrete had reached its destination did not show any appreciable difference, whether the concrete was dumped from the mixer as soon as it was possible to discharge it after loading or whether it was mixed for 10 min., as it sometimes was due to delays on the job. My explanation of this at the time was that the drops, etc., in the distribution system acted as a mixer.

We made another series of tests in a different way. We stopped the mixer ¼ min. after it was loaded and took samples from different parts of the mixer, near the center and near each of the four sides. We repeated the experiment, using ½ min., ¾ min., 1 min., etc.

We found that there was a large difference in samples taken from different parts of the mixer for the short periods of time of mixing, and that as the time of mixing increased the difference decreased.

About the only suggestion that I have to offer is that an engineer should take into account how the concrete is to be handled after it leaves the mixer. I do not believe that the engineer should decrease the time of mixing as a matter of time-saving, as suggested. If the rest of the plant is capable of handling a given amount of material, the work would not be speeded up by a shorter time of mixing. If the rest of the plant is capable of handling twice as much, the same result would be obtained by having either two mixers or one twice the size. The mixer is often a small part of the total cost of a plant.

Azusa, Calif., May 27, 1933. DANIEL MCFARLAND, Civil Engineer.

* * *

Sir—When the article on time of mix was published, Feb. 9, I expected a refutation of the statement on the part of research workers in that field, for I was filled with consternation at the unqualified statement that mixing past the 60-sec. period is worthless. We who write specifications for time of mix always have in mind that the time of mix will be cut down at various times to enable the contractor to make his planned pour within the time limit set for his labor shifts, and for that reason we issue, in addition, instructions to concrete inspectors that include a minimum time of mix for that purpose, which is usually set at 1 min. If this minute time of mix were adopted for specification purposes, the time of mix would surely be cut down at the slightest provocation, and to as low as ½ min. in emergencies. The writer, on innumerable occasions, even where there was fairly efficient inspection and a conscientious contractor's superintendent on the work, has found an over-zealous man at the mixer discharging his mixer within 30 sec. or less after the water was added to the mix. It isn't possible on most jobs to have an inspector at each mixer and at each point of placing concrete, and these conditions are bound to exist even with time-lock devices.

Now as to the physical points involved: I'm afraid that Major Besson did not consider all the factors involved in the discussion of time of mix. One of these is the use of light-weight aggregate such as light slag or haydite. Anyone watching the mixing of concrete with this type of aggregate would be convinced simply by visual inspection that the concrete mixed 1 min. is nowhere nearly a homogeneous mass, which this mixture should be at the end of this period of mix. Such a mixture (haydite) at such a time, when discharged from the mixer and placed in the forms, would have most of the coarse aggregate floating on top. As a matter of fact, haydite concrete requires more nearly 2 min. for mixing than 1 min. Another factor, I believe overlooked by Major Besson, is the size and shape of the mixer, and this factor is becoming increasingly important with the prevalence of central mixing plants and so-called transit-mix, where 4, 5 and 6-cu. yd. batches are delivered on the job. Unless properly mixed, they will have all sorts of varying consistencies in the various sections of mixer or truck, when delivered by the chute for placing in the forms. The writer has observed such variations on several occasions and has felt inclined to discourage the use of such large mixers on the job.

While there is no question that the greatest benefit from mixing occurs up to the end of 1 min., the writer refuses to believe, based on innumerable observations on any number of jobs, that there is no benefit from mixing after 1 min., though free to admit that the benefit after 1 min. is not nearly commensurate with that of the first minute.

In conclusion, I wish to urge specification writers on concrete work not to restrict their time-of-mix provision to 1 min., because the factor of safety for that feature of concrete-making is reduced to one, which is much too small. While D. B. Steinman has rightfully named the factor of safety a "factor of ignorance," it is in this case a very necessary factor because of the ignorance of the specification writer of what may happen in the field, of the efficiency, thoroughness and rigidness of inspection, of the shrewdness of the over-zealous operator of the mixer, of the lack of omniscience of the superintendent of construction and of the owner's engineer.

Hoboken, N. J., May 18, 1933. M. HIRSCHTHAL, Concrete Engineer, D., L. & W. Railroad.

Areal Subsidence

Sir—In your issue of June 29, 1933, p. 845, there is an interesting article on recent areal subsidence found in releveling in the vicinity of San Jose, California. The writer has been interested in water conservation in this vicinity since 1920 and believes it probable that the subsidence was caused by the decline of the water plane.

Santa Clara Valley is a very highly developed horticultural section irrigated by over 2,000 wells. In 1920 the well withdrawals exceeded natural replenishment, and a permanent decline of the water plane set in. The average water level has declined about 100 ft., in places as much as 200 ft. Over large areas the water plane is now considerably below sea level. The northern end of the Santa Clara Valley, particularly in the vicinity of San Jose, under natural conditions was an artesian area. Water-bearing gravel strata encountered at depths of 100 to 200 ft. were overlaid with impervious marine clays. When penetrated by wells, the water would rise 10 to 20 ft. above the ground surface and would flow continuously. The change of volume in the formerly saturated clay, which has now been drained, should account for the settlement that was noted at various points, reaching a maximum at San Jose and seeming to be roughly proportional to the decline in the water plane at the different points. I note particularly that the subsidence disappears at Coyote, about 15 miles south of San Jose. This point is on the gravel delta cone of the largest stream in this vicinity and is at a point where there is no important drop in the water plane.

Even the apparent acceleration of the subsidence during the last two years at San Jose synchronizes with the maximum rate of decline of the water plane for the very dry season of 1931-'32.

San Jose, Calif., July 19, 1933. FRED H. TIBBETTS, Chief Engineer, Santa Clara Valley Water Conservation District.

Is Time the Essence of a Contract?

Sir—One of the most stressed items in public and private contracts—the time-of-completion clause—might properly come up at this moment for discussion when steps are being taken to launch the country into another era of construction activity. With the new deal in so many things it might not be amiss to consider a new deal in time clauses, for a not too close inspection of work completed in the past era will

reveal that this clause might be revised in the interest of the builder and the owner.

The construction industry might well be as much ashamed of its past record of the boom era as the investment bankers must be of theirs; the lack of conservatism most notable in banking fields finds a deadly parallel in the construction field. For every issue of securities in the boom era that was sold on the basis of big returns and little safety, there have been thousands of dollars worth of public works let and built on the basis of "how quick," not "how well." And if as rigid an investigation were made of construction projects with a view to determining quality as was made of banking issues to determine safety, the construction industry would be equally, if not more thoroughly discredited.

Time became not only the essence of the contract but the only qualifying clause in no small number of cases as to the qualifications of the builder. Many contractors were awarded work on the basis of the least number of calendar days or working days bid, where the time of completion was not stated in the proposal but left to be put in by the bidder. Each day was evaluated on a liquid-damage basis, and the successful bidder was so determined. In some sections the calendar day crept in, and it did not make much difference whether the structure was to be started in January or June, because the calendar days, neglecting any consideration of the season, were the same.

When the contract time for a structure is short, the effect of the short time is bound to be reflected in the character of the workmanship. Under pressure of time the builder rushes extra equipment to the job at the start. Some of it is used only a small part of the time, which is a waste of plant. Large numbers of laborers are hired, more than likely an inefficient crew, just for the sake of getting started. The engineer is rushed for stakes, and some mistakes are made; pleas are made to waive technicalities so that the first materials, shipped in a hurry and not quite up to specifications, will not be rejected. The cry is "Don't hold up the work!" The shortness of time is felt throughout the job, and no one is immune, much less the engineer and contractor. Labor is affected; stone-setters, marble men, etc., are hired on their own say-so. The contractor depends on the regular men to keep the work up to standard, but volume must be produced, and so the regulars go over the job, pointing up, rubbing, covering up the defects of the work of the extras who were put on to save time.

And who is responsible? Projects that have been discussed for one, two or even three years, and for which plans have been ready for over a year, are advertised for letting ten days after the first notice appears in the papers, and the contractors line up to say how quickly they will complete them, or perhaps they are arbitrarily given 100 days to finish. The engineers set the time and make it as short as possible, so that the members of their own profession who are in the field will have as little employment as possible. At least one would think that was the occasion for the rush.

And to what carelessness has this time element led us? One need not go far to find instances. Even Washington, D. C., has glaring examples. There you will find monuments erected for the ages, and not yet ten years old, showing carelessness, defects of materials and workmanship. A marble column plastered to cover up a defect the size of a fist, a cracked balustrade, a craze-marked terrazzo floor—all bespeak the cheapness of the American people with their time.

Inspect the marble work in some of our finest buildings, marvel at nature for giving us such magnificent materials, and defend if you can the rush that causes them to be so badly abused in their use. Chipped, plastered, cracked, they are true examples of the craftsmanship of the time. And then remember that it takes time to assemble materials and craftsmen—but we seem never to have had the time.

Contractors differ not greatly from individuals in other walks of life. The depression has exposed nothing more amusing to a contractor than the fact that his field of business harbored its share of intangibles in the form of skimpers. But there are not many contractors, proud though they may be of their work, who cannot stand off and gaze at their masterpieces, and deep down in their hearts admit that if they but had more time their work would have been the better for it.

Dallas, Tex.,
June 24, 1933.

A CONTRACTOR.

Concrete or Timber?

Sir—Can a structure be guilty of deceit? With the specious plea that the "timber-seeming" bridge described in your issue of Aug. 3, 1933, p. 140, "harmonize with its wooded surroundings," the designers have produced a structure palpably dishonest. Why, if wood was the desired medium, did they choose concrete? Evidently they felt that timber construction would result in higher maintenance costs, or that the 62-ft. main span would have been a severe tax on wood construction. It would seem obvious, then, that some other medium must be chosen; in this case, reinforced concrete. Therefore, if concrete seems best to fulfill the structural conditions imposed by the site, it is then really "harmonious" and deserves treatment in its own right.

But of course the bridge is not entirely bad. A quite similar structure (but a more honest one) would have been evolved if the designer had laid down a five-point platform (paralleling that of the author's) such as this:

1. The girder section at mid-span is to be reduced, maximum section being provided at the supports where maximum shears and moments occur.

2. Iron brackets and bolts are to be used in the hand-rail, for these provide a good means of joining the respective members. [This is, of course, questionable.]

3. Slenderness, rather than massiveness, is to be sought in the members making up the hand-rail.

4. The hand-rail is to be chamfered. [This is reasonable in cast work.]

5. The exposed concrete is to be stained, say, a brown color. [There is no inherent falsity in this.]

EDWARD WILLIAM BENES.

Mt. Vernon, N. Y.,
Aug. 5, 1933.

Cold-Driven Rivets

Sir—In connection with the article on cold-driven rivets (*ENR*, June 15, 1933, p. 784) and the letter of July 27, 1933, p. 113, it might be interesting to call attention to some tests of rivets in tension that were carried out in the engineering experiment station of the University of Illinois. The results of these tests have been published in Bulletin 210 of that station.

In this series of tests one of the variables studied was the effect of the driving temperature upon the tensile strength of the rivet, and some nine specimens were tested in which the rivets had been driven cold with a bull riveter. All rivets had a $\frac{3}{4}$-in. diameter with two button heads and a grip of 2 in. Some trouble was experienced in driving due to the tendency of the shank to bend when the pressure was applied. This difficulty was overcome by using a shank shorter than the standard for a $\frac{3}{4}$-in. rivet with a 2-in. grip and by using a worn $\frac{5}{8}$-in. die in forming the head. This last made the head somewhat smaller than usual.

The rivets were not as tight as those driven while hot, as shown by the low values for initial tension obtained—initial tension being one of the factors determined in the tests. However, in all nine of the cold-driven specimens the ultimate tensile strengths of the rivets were greater than the ultimate tensile strengths of the rods from which the rivets had been made and were also greater than the ultimate strengths of similar hot-driven rivets. The average ultimate strengths of three groups of three specimens each were 63,800, 84,500 and 78,630 lb. per sq.in. In computing these figures it was assumed that the rivet filled a $\frac{13}{16}$-in. diameter hole. The high strengths obtained from these rivets made from ordinary rivet steel were due to the cold-working of the metal in driving, and they at least show that cold-driven rivets are adequate in tension.

WILLIAM A. OLIVER,

Urbana, Ill.,
July 31, 1933.

Associate in Civil Engineering,
University of Illinois.

Engineering News-Record

EDITOR—F. E. Schmitt. EDITORIAL STAFF—V. T. Boughton, W. G. Bowman, F. W. Herring, C. S. Hill, H. W. Richardson, W. W. De Berard, N. A. Bowers, J. I. Ballard.

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Road Contracts Being Let

CONTRACTS are at last being awarded for road construction, it is gratifying to note. Some of the $400,000,000 provided for highway improvement in the National Industrial Recovery Act is thus beginning to filter into the arteries of trade, and more men are getting onto the payrolls of construction. This is the accomplishment that the framers of the Recovery Act had in mind when they voted in June last to spend unprecedented sums for public works, and it is the initial stage of the action on which the construction industry has pinned its hope for starting up general economic revival through rehabilitating the millions of workers who draw their living in the contracting, materials, machinery and transportation businesses. The count of operations so far begun is not large, but it is the beginning of the rain always that signifies the breaking of the drought; full streams and a revived land come as the rain continues. The present indications are that scattering announcements of roadbuilding operations already started will accumulate fast into steady streams of work in progress. There is much hope both for industry at large and for the highway industries in particular in the news that contracts for roads are being awarded.

Circular No. 2

THE Public Works Administration is to be commended for its oft-stated desire to see that none of the money provided by the Recovery Act is squandered needlessly. In the writing of Circular No. 2, however, outlining the information to be furnished by states, cities and other municipal bodies applying for loans, the Administrator appears to have gone to quite unnecessary extremes in setting up requirements. The money provided by the act will be effective only if put into circulation quickly and scattered widely through relatively small projects in many communities. Applications for funds for such projects must come voluntarily and quickly. But they will not come voluntarily or quickly if many weeks must be spent in collecting and compiling a great mass of legal and financial data. When a mere listing of the requirements in small type requires well over one of our pages, it is easy to imagine the amount of work required to compile and present the data. Hundreds of communities are planning to apply for loans under the act; each week our construction news pages carry many such announcements. Many of these communities have become impatient over the long delay in organizing state and regional public-works boards to the extent that their interest in the work has waned. If it appears now that the Public Works Administration is going to adhere to the letter of its requirements as set out in Circular No. 2, many of the projects now proposed will never reach the application stage. Some statement of intent and of desire for quick action should come from the Administration if the bad effect of Circular No. 2 is to be counteracted.

Inconsistent

CAREFUL STUDY of Circular No. 2, "Information Required With Applications for Loans to States, Counties, Municipalities and other Public Bodies," suggests that it is regrettable that the Public Works Administrator waited to issue the circular until after $63,000,000 had been allocated to the state of Washington for the start of work on the Columbia Basin project and $22,700,000 to Wyoming for the Casper-Alcova project. If the advocates of either project had been required to give satisfactory answers to the multitude of questions contained in Circular No. 2, no money would have been forthcoming. If the small community that is asking for $100,000 for a new water-supply system is to be required to answer the long catechism outlined in Circular No. 2 in order to prevent wasteful expenditure of government funds, surely the applicants for these $63,000,000 or $22,700,000 allocations might well have been required to do likewise.

Worth the Cost

FEW DATA have been available to substantiate the general assumption that regulations and restrictions placed upon public-works construction by the National Recovery Act will increase the cost of work. In this light the bids submitted for the substructure of the Little Bay Bridge near Portsmouth, N. H., are of interest. Regular proposals were asked for on the basis of state financing, with no restrictions as to wages and hours of labor. Before bids were received, supplemental proposals were requested, naming a lump-sum increase over the original bids if the project were to be placed under emergency federal financing with regulations as to working hours, prevailing wages, local preference of labor, etc. The increase named by the ten bidders ranged from 1.8 to 9.9 per cent of the original bids, the average being 5.8 per cent. While these increases are only estimates and are not actual cost data, they indicate that the public will not be heavily penalized by excessive construction costs under the program of re-employment and business recovery. Six per cent is a small premium to pay for any plan that will increase employment and raise wages.

Two Who Made History

FORTY YEARS AGO two men were engaged in engineering studies in the West that were destined to open a new chapter in the nation's history. The investigations that Frederick H. Newell and Arthur P. Davis carried on for the U. S. Geological Survey of water resources in the regions of the great deserts laid the foundation on which later was framed the epoch-making program of federal land reclamation, itself part of that broad policy of conservation of natural resources that was championed by Theodore Roosevelt when he became President, with the aid of such men as Senator Newlands and Gifford Pinchot. Frederick Newell led in the strategy and fighting that put the Reclamation Act on the statute books in 1902. If only for this, he would long have a place of note in our record of engineering achievement; but when, upon passage of the act, he became director of the newly organized Reclamation

Service he did no less constructive work in building up the remarkable body of engineers that distinguished the Service. Arthur Davis became his principal assistant engineer at the very beginning, and together they successfully wrestled with the many intricate problems of political, human and technical kind that the new enterprise brought to the fore. Thus, the brilliant achievements of reclamation stand largely to the credit of these two men. That both of them were repaid by governmental ingratitude—Newell was discharged by Franklin K. Lane, Davis by Hubert C. Work, holders of the notoriously political secretaryship of the Interior Department—merely increases their claim to recognition by the engineering community. The spirit and the policies which they brought into being still exist, and, though changed economic conditions now raise critical question about the basic soundness of the federal reclamation enterprise, the work that they did in causing some millions of acres of desert to flourish and blossom will long endure.

New Plans for the Mississippi

THE PRESENT PROGRAM of flood control on the Mississippi River is a noteworthy attempt to place this century-old undertaking on a national basis. As described in the series of articles concluded last week, the change marks a new era in Mississippi River engineering. It is not unreasonable to believe that it may influence generally the practices of improving alluvial rivers. Specifically, tradition and opinion, which have ruled for half a century on the Mississippi, have been swept away by research and invention. Efforts piously persisted in, all this time, to modulate the dynamic forces of the river have been replaced by a decision to put these forces to the work of self-regulation. The change in policy and program is epochal and merits unusual consideration.

With a tortuously shifting course and an irregular longitudinal profile, the Mississippi River is an indifferent hydraulic conduit. This characteristic has not gone unrecognized in the past, but such recognition was rendered sterile by the long-held belief in the hydraulic sanctity of the existing regimen. Improvement of the river as a conduit for the disposal of water could make no progress under these conditions. Efforts were centered on preventing overflow due to conduit deficiencies from overstepping assigned lateral limits. This policy was expressed in persistent doctrine of "Levees Only." Experience, it should be said, had given much justification for this dogmatic decision, for in most instances where in past times curtailment of channel had occurred the river had taken back the mileage and sinuosities of which it had been robbed.

Refusal to continue to accept old experience and opinion as conclusive, constitutes the fundamental difference in the new thinking. As the problem appeared, the critical section of the river was between the mouth of the Arkansas River and the mouth of the Red River. Above the Arkansas the flood problem is solved by levees, and below the Red it is solved by a combination of levees with the Atchafalaya and Bonnet Carré floodways. Between these stretches the only possible solution was greater channel capacity if the legally embroiled and costly floodway through the Boeuf-Tensas basin was to be avoided. Study indicated that a more capacious channel was possible if the flat profile below Natchez could be continued upstream to Vicksburg, which might be accomplished by cutting off bends and removing the dam caused by the change in bed slope at Natchez. Such procedure, however, involved a drastic change in existing channel regimen, which in view of past belief and experience could not be undertaken lightly. Research, the second element of the new planning, was therefore called in to support invention.

Beginning almost with its completion, the Waterways Experiment Station established at Vicksburg was given the task of determining the effects of channel-shortening by cutting off bends. Research was then carried into river-bottom materials, bed-load movement, erosion and sedimentation, and finally extended to broader laws of river flow and the operation of all the factors that influence flow. This work, still going on, appears to be evolving cumulatively a new science of alluvial river action. As experimentation has reached results, they have been applied to the river operations; the four artificial cutoffs on the Vicksburg-Natchez section and others on the Atchafalaya are examples. Experimentation and trial also are leading toward culmination in the great undertaking of smoothing out the bed profile and flattening the slope from Vicksburg south, a fundamental element of the new plan. All other operations—control of the Red-Atchafalaya-Old River combination of channels, cutoffs, diking and revetment—are contributory elements to this slope regulation, which latter represents the outstanding creative development and construction undertaking of the new plans for the Mississippi.

The processes of accomplishing the change in slope evidence invention as bold as that which conceived its undertaking. Briefly, the hard spot that causes the break in the bed slope at Natchez (and possibly other up-river hard spots) is to be cut through by dredging. With the detritus-carrying capacity of the river increased by preceding cutoffs, agitation dredging is to continue during rising and falling stages. This process consists essentially of dredging up the bottom and immediately discharging the dredged material and pump water overboard, where the river current will carry it away by suspension and bottom drag. In principle the dynamic forces of the river, emphasized by channel-deepening and straightening, are to be set to work to clear and enlarge its own channel with the aid of bottom agitation by dredging. The operation is the application on a grand scale of procedure that has in lesser operations proved successful in channed enlargement.

As to the success of the plans as reviewed here and described in the articles published during the last two months, probably no one is prepared to make a positive pronouncement. So far as evidence concerning the cutoffs is available thus early in their life, no untoward results are indicated. It seems unlikely that any results will develop beyond the power of revetment and contraction works to handle, and both these auxiliary devices are contemplated in the broad plan, subject to the showing of trial and experience.

Considered technically, the new plan deserves express recognition for the courage reflected in its experimentation, fundamental research and practical large-scale application. If it is successful, it will be a triumph over conditions that have challenged engineering skill for fifty years. Meanwhile, the lessons being learned from experimentation are contributing largely to the knowledge of the hydraulics of sedimentary rivers.

NEWS OF THE WEEK

Important Changes In Construction Code Submitted to NRA

CHANGES in the Code of Fair Competition for the Construction Industry from that submitted to the National Recovery Administration on Aug. 7 are included in a new draft submitted to the NRA on Aug. 11. The changes affect the sections relating to minimum wages and maximum hours, also local agreements concerning hours and wages. They are covered by new sections 3, 4 and 4a to be substituted for sections 3 and 4 in the original draft as published in *ENR* Aug. 3, p. 148.

Modify blanket agreement

The proposed changes in the code for the construction industry are of immediate importance to the construction industry because they, together with Section 9, were accepted by the Administrator on Aug. 11 as substitutes for sections 2, 3, 4, 5, 6 and 12 of the President's Reemployment Agreement (so-called President's Blanket Code) for all employers coming within the definition of the construction industry as defined in Section 1 of the construction industry code. Accordingly, such employers may sign the PRA with these sections substituted using certificates of compliance obtained from local post offices.

The code submitted by the League fixes a minimum rate of wages for unskilled labor of 40c. an hour, a maximum average of 35 hours a week during a 6-month period, and an absolute maximum in any one week of 48 hours or 8 hours a day. A 40-hour week is established for accounting, clerical and office employees whose minimum rates of wages are as follows: $12 per week in towns of less than 2,500; $14 per week in cities between 2,500 and 250,000; $14.50 in cities of 250,000 to 500,000; and $15 in cities over 500,000.

Changes in the code

Deleted from the code as finally submitted to NRA was a provision, coupled with the 40c. minimum, that wages shall be "not less than such minimum rates as may be established nationally or for a region or locality by mutual agreements reached through bona fide collective bargaining between truly representative national, regional or local groups of employers and employees." The same provision with respect to hours also was eliminated.

Hours and wages fixed by existing contracts will not be disturbed. This exemption was added to the provision exempting from the code employees whose wages and hours of labor are established for specific projects by competent governmental authority.

Contractors are bound by Section 7 of the President's re-employment agreement not to reduce the compensation of employees receiving more than the minimum

(Continued on page 211)

Private Loan Requirements Outlined in NRA Circular No. 3

Information that will be required of private corporations applying to the National Recovery Administration for loans for the construction of privately-owned public service projects under the provisions of the Recovery Act is outlined in Circular No. 3 published by the administration on Aug. 15.

In general the requirements are similar to those set out for projects to be constructed by public bodies in Circular No. 2, issued last week (*ENR*, Aug. 10, p. 182). Applicants are required to give complete information as to their organization, history, financial and legal set-up. They also must include a general description of the project with cost and time required for construction, its relation to like projects and to a general plan for the development of the region, expected revenues and method of financing. All objections to the project and the names and addresses of all objectors must be listed.

Contracts Get Under Way On Federal Highway Work

Approval of the assignment of federal road funds for Maryland completes the road apportionment for all states with the exception of Georgia, where local differences between the governor and the state highway department have held up the program.

The Public Works Administration announces that 300 public road projects have cleared both the PWA and the United States Bureau of Public Roads. These projects are located in 185 counties in 19 different states and the District of Columbia.

Reports from district engineers of the Bureau of Roads show 15 projects already under construction. Work on at least 15 others is expected to get under way immediately.

Program maps for highway work had been approved for 36 states and the District of Columbia on Aug. 11.

State Engineers for Public Works Announced by Secretary Ickes

APPOINTMENT of state engineers for the Federal Emergency Administration of Public Works was announced on Aug. 1 by Harold L. Ickes, Administrator.

The state engineers are federal agents responsible to the Administrator. They will serve as executive officers for the state advisory boards, organize the state offices, direct the personnel, receive, record and examine all applications. They will report their findings to the state advisory boards which will make recommendations to the Administrator at Washington.

Together with the state boards the engineers will prepare preliminary state programs which will include projects upon which work can be started immediately.

The projects which will be included in this preliminary schedule will be examined by the engineers from the viewpoint of soundness, of engineering plans and design, economic and social necessity, financial feasibility and other required qualifications as set forth in the circulars already issued by the Public Works Administration.

Headquarters of the state advisory boards and the state engineers will in most instances be in free space provided by the states. Most of the state governors responded immediaately to the Administrator's call to make space available.

As a rule the headquarters are in the state capitals, but in some instances, where local conditions make other cities more suitable and convenient, those cities will be designated as headquarters. These are to be announced later.

In several states qualified men who are members of state advisory boards were transferred to become state engineers for the Public Works Administration. Vacancies thus created on state advisory boards will be filled later.

A list of state engineers of public works, with some appointments still to be made, follows:

Alabama—Armour C. Polk, Birmingham.
Graduate in civil engineering of Virginia Military Institute 1899, Rensselaer Polytechnic Institute, 1903. An executive of engineering and construction companies, has recently had charge of construction throughout Southeastern United States.

Arizona—Howard S. Reed, Phoenix.
Consulting civil engineer engaged in general practice in Arizona and New Mexico. Captain in World War with U. S. Engineers. Former City Engineer, Phoenix, Arizona.

Arkansas — Alexander Allaire, Washington, D. C.
Graduate of Stevens Institute of Technology. Experienced in general construction, having been in active practice with large companies for twenty years.

California—Frank E. Trask, Los Angeles.
Consulting engineer specializing in water works, irrigation, valuation and rate cases.

Colorado—George M. Bull, Denver.
Graduate of Rensselaer Polytechnic Institute. Specialized in water supply and irrigation for past 10 years. Has been Chief Engineer for Denver City Water Board.

Connecticut—Leslie A. Hoffman, Bridgeport.
Graduated from Massachusetts Institute of Technology 1917. Recently engaged in conception and execution of industrial projects involving steel and mill construction and heavy foundations.

Delaware—Charles H. Fleming, Dover.
Civil engineer, engaged in municipal water supply construction and operation.

Florida—James E. Cotton, Miami.
Graduated from Iowa State College in civil engineering. Has recently been engaged in construction of wharves, docks, municipal buildings, in South Florida.

Georgia—J. Houston Johnston, Atlanta.
Graduated from University of Virginia in civil engineering. Recently in private practice as consulting engineer in the state of Georgia.

Idaho—Ivan C. Crawford, University of Idaho, Moscow.
Graduated 1912 from University of Colorado in civil engineering. Has been Dean and Professor of Civil Engineering at the University of Idaho since 1923.

Illinois—Wilfred W. DeBerard, Chicago.
Mr. DeBerard is western editor of *Engineering News-Record* and a sanitary hydraulic, and municipal engineer, recently a member of the Regional Planning Association of Chicago.

Indiana—Albert H. Hinkle, Indianapolis.
Graduated from the State University in civil engineering, 1907. Specialized in highway construction and maintenance.

Iowa—P. Frank Hopkins, Mason City.
Civil engineer, specializing in the design and construction of municipal utilities. Recently City Manager of Mason City.

Kansas—Robert J. Paulette, Salina.
Consulting civil engineer. Graduated University of Missouri 1912. Has recently specialized in municipal construction, at present is a consulting engineer.

Kentucky—Robert V. L. Wright, Washington, D. C.
Attended University of Maryland and Columbia University. Consulting engineer experienced particularly in municipal, public utility, and railroad projects.

Louisiana—Orloff Henry, New Orleans.
Graduated from Tulane University, 1906. Was recently engaged in private practice as a consulting engineer.

Maine—George M. Williamson, Washington, D. C.
Graduate of Cornell University, special student at Colorado School of Mines. Has recently engaged in consulting practice in Southwestern United States.

Maryland—Abel Wolman, Baltimore.
Consulting engineer. Graduated from Johns Hopkins University, 1915. Has recently been chief engineer, Maryland Department of Health.

Massachusetts—Charles R. Gow, Brookline.
Mr. Gow is president of Warren Bros. Co., is a former postmaster of Boston, an ex-president of the Boston Society of Civil Engineers and of the Associated Industries of Massachusetts. He has been a member of several engineering commissions in New England.

Michigan—Mortimer E. Cooley, University of Michigan, Ann Arbor.
Formerly Dean of College of Engineering and Architecture, University of Michigan. Graduate of U. S. Naval Academy and University of Michigan. Past Chief Engineer U. S. Navy, consultant on municipal railroad and public utilities work.

Minnesota—William N. Carey, St. Paul.
Mr. Carey is a consulting engineer carrying on a general practice covering municipal, railroad, river and harbor developments, city planning and public works.

Mississippi—George H. Wells, Washington, D. C.
University of Virginia. General engineering experience, covering design, and construction. Recently a member of the staff of the Cincinnati Union Terminal Company.

Missouri—Hugh Miller, Washington, D. C.
Graduate Princeton University, 1901. Formerly Dean, College of Engineering, George Washington University, Washington, D. C. During War, Captain Corps of Engineers, U. S. Army, now Lt.-Col. Reserve Corps.

Montana—Donald A. McKinnon, Washington, D. C.
Graduate in civil engineering, University of South Dakota. Formerly engineer with the Highway Department of South Dakota, more recently a member of engineering and contracting firm engaging in highway and municipal work. Served as Captain of Field Artillery in the A.E.F.

Nebraska—Albert C. Arend, Omaha.
Graduated from Rutgers College in civil engineering, 1892. Recently engaged in general consulting practice in industrial and public utilities and river channel protection. One time City Manager of Omaha.

Nevada—
State engineer of the Public Works Administration for Nevada will also serve for Utah. See Utah and Nevada.

New Hampshire—Harold J. Lockwood, Dartmouth College, Hanover.
Professor of Power Engineering at Thayer School, Dartmouth College, and has engaged recently in a general consulting practice.

New Jersey—Abraham B. Cohen, South Orange.
Graduate of Purdue University in civil engineering. Consulting engineer specializing in design and supervision of construction on viaducts, bridges, concrete structures, planning and grade crossing eliminations.

New Mexico—
The State Engineer of the Public Works Administration for New Mexico will also serve for Arizona—see Arizona.

New York—Appointment to be announced later.

North Carolina—Herman G. Baity, University of North Carolina, Chapel Hill.
Dean of Engineering School, University of North Carolina. Active in engineering educational work in North Carolina Association. Sanitary engineer North Carolina State Board of Health.

North Dakota—Harold C. Knudsen, Dickinson.
Connected with the State Highway Department of North Dakota for a number of years.

Ohio—William A. Stinchcomb, Cleveland.
Civil engineer connected with public service work in Ohio. Since 1920, has been engineer consultant for the Cleveland Metropolitan Park System.

Oklahoma—Philip S. Donnell, A. & M. College, Stillwater.
Graduate Clark University and Harvard University. Connected with U. S. Coast & Geodetic Survey. Formerly Professor and Dean of Engineering, University of New Mexico. At present, Dean of Engineering, University of Oklahoma. Served as officer in World War.

Oregon—Claude C. Hockley, Portland.
Consulting engineer. Private practice. Graduate of Ohio Northern University in civil engineering. Formerly identified in a paper manufacturing industry and as consulting engineer. Has specialized in hydro-electric developments and in pulp and paper investigations.

Pennsylvania—William H. Gravell, Philadelphia.
Specialist in building, construction and Major Corps of Engineers in A.E.F.

Rhode Island—
The state engineer for the Public Works Administration for Rhode Island will also serve for Connecticut—see Connecticut.

South Carolina—J. L. M. Irby, Cleveland.
Graduate of Citadel College. Has been identified in engineering work in the development of lumber industry. Served as Captain of Engineers in World War.

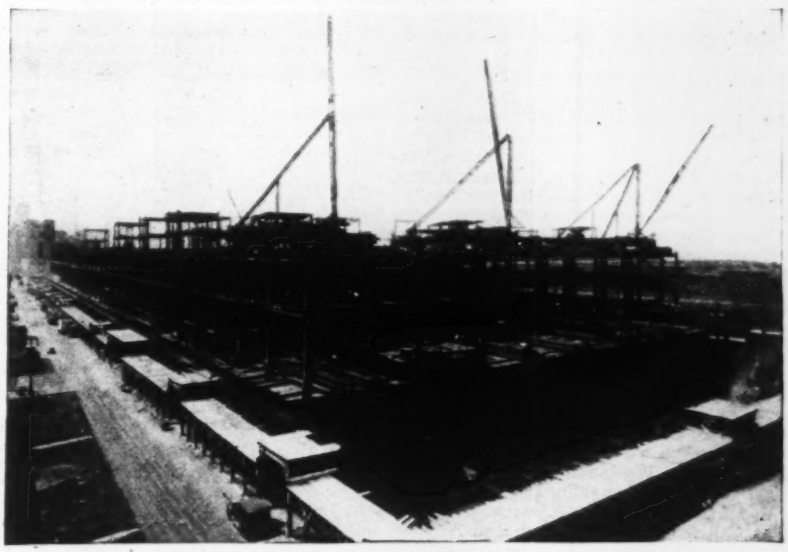
South Dakota—
The state engineer for Public Works for South Dakota will also serve for North Dakota—see North Dakota.

Tennessee—Harry S. Berry, Nashville.
Mr. Berry is a West Point graduate and served in the army in Philippines, Mexico and overseas. Formerly a State Highway Commissioner.

Texas—Robert A. Thompson, Dallas.
Consulting civil engineer. Graduate of University of Texas, civil engineering and M. A. Formerly in railroad work and member of Engineering Board Interstate Commerce Commission on railroad valuation. In charge of irrigation and reservoir work throughout Southwest. Formerly Chief Engineer of State Highway Commission.

Utah and Nevada—Richard A. Hart, Salt Lake City.
Graduate of University of Utah in civil engineering. Specialist in drainage and irrigation projects, and author of papers on those subjects. Recently engaged in Chamber of Commerce Work.

Vermont—
The state engineer for Vermont will also serve for New Hampshire—see New Hampshire.



STEEL FRAME OF NEW YORK CENTRAL FREIGHT TERMINAL IN NEW YORK COMPLETED

On July 26 the last of the 21,000 tons of steel in the frame-work of the huge freight terminal building of the New York Central Railroad in lower Manhattan was erected. Although designed for 12 stories, only 4 stories of the building will be erected at present.

The building is a part of the New York Central grade elimination work on the west side of Manhattan and forms the southern terminus of the depressed and elevated line that is being built south from the company's 60th St. yards. Tracks enter the building on the second floor alongside platforms equipped with elevators, providing connections with storage space on the upper floors and trucking bays at street level.

The building is known as the St. John's Park Freight Terminal although it is located several blocks north of St. John's Park, the site of the present terminal that is to be abandoned. The new building is 797 ft. long parallel to and fronting on West Street, a wide thoroughfare that serves the Hudson River piers. The building is 219 ft. wide at one end and 276 ft. at the other.

The structure was designed by the engineers of the New York Central Railroad and is being built by James Stewart & Co., New York, with Post & McCord as steel erectors.

Virginia—James A. Anderson, Virginia Military Institute, Lexington.

Professor Civil Engineering V.M.I. Graduate of Virginia Military Institute and Cornell Universities. Has been identified with engineering educational work for a number of years. Served as Captain and Lt. Col. in World War.

Washington—Gene Hoffman, Olympia.

Has been prominent in state highway work in Washington for a number of years.

West Virginia—M. Lindsay O'Neale, Fairmont.

Graduate of Columbia University in mining engineering. Chief engineer and safety director of coal company. Has been identified with the mining industry of West Virginia as consultant and as an operating official.

Wisconsin—James L. Ferebee, Milwaukee.

Graduate North Carolina State College in engineering. Has specialized in sewage disposal and water supply for municipalities. Recently chief engineer sewage commission of Milwaukee.

Wyoming—Francis C. Williams, Sheridan.

Graduate of Massachusetts Institute of Technology. Has been in private practice in Wyoming specializing in railroads, water supply, sanitary and irrigation work.

Ten Regional Inspectors Appointed to PWA

Appointment of ten regional inspectors of the Public Works Administration was announced on Aug. 15. They will have the duty of assuring that contract specifications are lived up to and that government funds are dispersed as intended. They will receive their orders and report directly to the Public Works Division of Investigations in Washington, which is under the direction of Louis R. Glavis, who also is head of the Division of Investigations of the Department of Interior. Investigators in each region will investigate complaints filed with the regional inspector.

The regional inspectors, together with their headquarters, are as follows:

D. J. Barry, Young's Hotel, Boston, Mass. The region consists of Maine, Vermont, New Hampshire, Massachusetts, Rhode Island and Connecticut.

Wharton Green, Federal Building, New York City. The region consists of New York, New Jersey and Pennsylvania.

Charles Allen, Old Postoffice Building, Chicago, Ill. The region consists of Illinois, Indiana, Michigan, Ohio and Wisconsin.

Robert A. Radford, Postoffice Building, Omaha, Neb. The region consists of North Dakota, South Dakota, Nebraska, Minnesota, Iowa and Wyoming.

Roy F. Bessey, Postoffice Building, Portland, Oregon. The region consists of Montana, Idaho, Washington and Oregon.

Frank Y. McLaughlin, Hughes Building, San Francisco, Calif. The region consists of California, Nevada, Utah and Arizona.

G. L. Rodier, Old Postoffice Building, Fort Worth, Texas. The region consists of Texas, Louisiana and New Mexico.

F. W. Kassebaum, Postoffice Building, Kansas City, Mo. The region consists of Colorado, Kansas, Oklahoma, Missouri and Arkansas.

C. L. Homer, Federal Building, Atlanta, Ga. The region consists of Mississippi, Alabama, Georgia, South Carolina and Florida.

F. R. Keyes, Postoffice Building, Baltimore, Md. The region consists of Tennessee, Kentucky, West Virginia, Maryland, Delaware, Virginia and North Carolina.

Information Needed For Housing Loans Listed by PWA

INFORMATION required by the Public Works Administration to be filed with preliminary applications for loans for low-cost housing or slum clearance projects is outlined in circular No. 4 issued on Aug. 11. The circular is available from the Superintendent of Documents, Washington.

The circular states that in order to save applicants unnecessary expense in preparing detailed plans, specifications and financial statements, only the information listed will be required for a preliminary application for a loan. If the examination of such a preliminary application indicates that the project is of a satisfactory character, the applicant will be encouraged to prepare the additional plans and data required for a final application.

This form of application is designed for private limited dividend corporations formed to undertake the particular housing project. Other applicants may be required to furnish some additional information.

Applications, together with supporting data, should be submitted in duplicate direct to the Administrator of Public Works, Division of Housing, Washington, D. C. They should not be submitted to State advisory boards.

The list of items of information required is as follows:

Information Required

I. The Applicant

(*a*) Name and address.

(*b*) Name, title, and address of official representative or representatives with whom correspondence should be conducted.

(*c*) Name and address of applicant's attorney.

(*d*) Name and address of applicant's architect and consulting engineer.

(*e*) Name and address of each officer and director of the corporation.

II. Construction Period

(*a*) Estimated time that will elapse after funds are allocated before—

(1) Work can be commenced.

(2) Project will be completed.

(*b*) Estimate of average number of men to be employed 30 hours per week directly on the project.

III. Character of Project

(*a*) *Site:*

(1) A map, showing the location and area:

NOTE — Physical features should be shown by two maps:

(I) Key map of city showing these facilities and proposed site.

(II) Sectional map showing these facilities in greater detail.

(2) A statement of the assessed valuation of the land necessary for the project and the established ratio between assessed value and fair cash value.

(3) Tax rate.

(4) Any unusual conditions adversely affecting the cost of the project.

(5) Description of present improvements, existing utilities, etc.

(6) Approximate number of present owners and proposed method of acquisition of site.

(7) Brief statement of advantages and disadvantages of the site for this particular project.

(8) Relation to planned growth of the city, employment and shopping centers, transportation facilities, schools, recreational facilities, etc.

(*b*) *Proposed improvements:*

(1) Character and type of building (fireproof, fire-resisting, ordinary construction, number of stories).

(2) Number of apartments or houses.

(3) Estimated number of persons to be housed.

(4) Ground coverage.

(5) Number and size of stores (if any).

(6) A sketch of the site layout and sufficient specifications and architectural drawings of a typical unit in sketch form to illustrate the type and character of the buildings contemplated.

IV. Management

A description of the proposed permanent operating and management organization.

V. Summarized Cost and Income Statement

(*a*) *Cost*—An estimate of the cost of the project summarized under the following items:

(1) Land (cost per square foot and total).

(2) Utilities, landscaping, streets, and other improvements.

(3) Buildings (cost per cubic foot and total).

(4) Architects' and engineers' fees.

(5) Preliminary and organization expense.

(6) Carrying charges during construction.

(7) Other costs.

(8) Total.

(9) Condensed estimates of the cost of labor and materials, stated separately.

(*b*) *Financing:*

(1) Amount of mortgage loan.

(2) Equity interest.

(3) Total.

(*c*) *Annual income:*

(1) Number of rooms and rental rate per room per month.

(2) Total annual rental.

(3) Linear feet of stores, rental rate, and total annual rental.

(4) All other annual income.

(5) Total gross income.

(6) Deduction of allowance for vacancies.

(*d*) *Annual expense:*

(1) Operating expense, number of rooms, and rate per room per annum.

(2) Taxes.

(3) Insurance.

(4) Total cost of operation.

(5) Balance available for interest and amortization.

(6) Interest on mortgage loan (assume 4 per cent).

(7) Amortization (assume a constant rate of 1.35 per cent on fireproof construction and of 2.4 per cent on ordinary construction).

(8) Total interest and amortization.

(9) Balance available for dividends and reserves.

VI. Planning and Related Data

(*a*) Statement as to whether or not project is part of a larger plan or long-range program of development and, if so, give brief description of other parts of plan or program.

(*b*) Statement as to whether or not there is a city or regional planning board and/or housing board, whether this project has been submitted to such board, and the views of such board, if any, in regard to it.

(*c*) Statement as to whether the community is part of a metropolitan district and, if so, whether the project is coordinated with the plans for metropolitan development.

(*d*) State whether the project has been approved by governmental or civic bodies and, if so, by whom. Give any other evidence of local sentiment favoring the project.

Public Works Administrator Ickes has announced that no consideration would be given to housing projects in which investigation disclosed evidence that the land required was being boosted in price on prospects of government loans for housing construction.

Robert D. Kohn, Director of Housing, has reported thirty housing projects from almost as many cities have been presented for consideration to the Administration. If approved, they will call for loans considerably more than $100,000,0000.

Among the schemes presented are several from metropolitan slum areas where high buildings to overcome high land costs have been proposed, Mr. Kohn said. He also stated that such projects are to be discouraged as unwisely permanently fixing high land values in slum areas. Such land values must reach a level of cost that reflects solely income possibilities for low-cost housing before it can be used for such purposes. The Housing Division advocates a limit of height for housing projects from five to six stories even in crowded centers.

Important Changes Made in Code For the Construction Industry

(Continued from p. 208)

wage rates, notwithstanding that their hours are reduced, and to increase the compensation of such employees by an equitable readjustment of all pay schedules.

Other codes rushed

With the general code for the construction industry submitted to the NRA, the code committee of the Construction League of the United States is devoting the current week to consideration of supplemental codes so that the principal ones—architects, engineers, general contractors and sub-contractors—may be submitted to the NRA at an early date. Deputy Administrator Muir has asked that these codes be submitted as the Administrator wishes to have the major supplemental codes considered at the same time that the general code is considered. A date for public hearing on the general code and supplemental codes will be set as soon as the principal supplemental codes are in the Administrator's hands. Conflicts between the supplemental codes are being adjusted by the code committee.

The code of the Associated General Contractors of America had its preview by the NRA on August 11 and some changes are now being made.

To date nearly 150 codes of some sort have been submitted to the NRA by national, regional or local groups representing both employers and employees in construction and related industries.

Code changes

The following sections of the construction industry code are those now substituted for corresponding sections in the code submitted to the NRA on Aug. 7:

3. *Minimum Wages*—Employers in the Construction Industry shall pay wages:

(a) Not less than the minimum rate of wages for unskilled labor hereby established which shall be not less than forty cents (40c.) per hour unless the hourly rate for the same class of work on July 15, 1929, was less than forty cents (40c.) per hour in which case the hourly rate shall be not less than that of July 15, 1929, and in no event less than thirty cents (30c.) per hour, and furthermore, in any event

(b) Not less than the minimum rate of wages for accounting, clerical or office employees hereby established as follows: $15 per week in any city of over 500,000 population, or in the immediate trade area of such city; $14.50 per week in any city of between 250,000 and 500,000 population, or in the immediate trade area of such city; $14 per week in any city of between 2,500 and 250,000 population, or in the immediate trade area of such city; and $12 per week in towns of less than 2,500 population. Population shall be determined by the 1930 federal census.

(c) Nothing herein contained shall be construed to apply to employees whose rates of wages are established for specific projects by competent governmental authority in accordance with law or with rates of wages established by contracts now in force.

4. *Maximum Hours*—Employers in the Construction Industry shall not employ any employee:

(a) In excess of the maximum average of thirty-five (35) hours a week during a six months calendar period, or forty-eight (48) hours in any week in such period, or eight (8) hours in any one day, excluding employees engaged in professional, executive, administrative or supervisory work; those engaged in cases of emergency work requiring the protection of life or property, and those in establishments employing not more than two persons in towns of less than 2,500 population, which towns are not part of a larger trade area. Population shall be determined by the 1930 federal census.

(b) Employers shall not employ accounting, clerical or office employees in any office or in any place or manner for more than forty (40) hours in any week, excluding employees in managerial or executive capacities and those in establishments employing not more than two persons in towns of less than 2,500 population, which towns are not part of a larger trade area. Population shall be determined by the 1930 federal census.

(c) Nothing herein contained shall be construed to apply to employees whose hours of labor are established for specific projects by competent governmental authority acting in accordance with law or with hours of work established by contracts now in force.

4-a. *Area Agreement for Hours and Wages*—Minimum rates of wages and maximum hours of labor may be established nationally or for a region or locality by mutual agreements reached through bona fide collective bargaining between truly representative national, regional or local groups of employers and employees. In no event shall such minimum rates of wages be less than those established in Section 3 hereof, nor in any event shall such maximum hours of labor be more than those established in Section 4 hereof, excepting such minimum rates of wages and such maximum hours of labor as are established for specific projects by competent governmental authority acting in accordance with law.

Pennsylvania Announces Policy Against Private Toll Bridges

In rejecting plans for a $1,200,000 bridge over the Schuylkill River, near Miquon Station, Montgomery County, Pa., the Public Service Commission of that state strongly enunciated a policy against the construction of privately owned toll bridges.

"This commonwealth has indicated clearly its adoption of a thoroughly crystallized public policy against privately owned toll highways, whether roads or bridges," the commission's order stated. "This policy is clearly established by statutory action and expenditure of public funds to eliminate toll roads and toll bridges."

Legislation provides for the freeing of the Philadelphia - Camden, Columbia - Wrighsville and other municipal toll bridges as soon as collections wipe out the cost. The state highway department is also empowered to purchase and eventually to free other toll bridges of the state.

SOCIETY CALENDAR

AMERICAN ASSOCIATION OF ENGINEERS, annual meeting, Chicago, Ill., Oct. 3-6, 1933.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual meeting, Milwaukee, Wis., Oct. 9-11.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, annual meeting, Chicago, Oct. 17-21.

AMERICAN PUBLIC HEALTH ASSOCIATION, annual meeting, Indianapolis, Ind., Oct. 9-12.

AMERICAN SOCIETY OF MUNICIPAL ENGINEERS, annual meeting, Milwaukee, Wis., Sept. 25-27.

AMERICAN WELDING SOCIETY, fall meeting, Detroit, Mich., Oct. 2-6.

HIGHWAY RESEARCH BOARD, National Research Council, Washington, D. C., Dec. 7-8.

INTERNATIONAL ASSOCIATION OF PUBLIC WORKS OFFICIALS, annual meeting, Milwaukee, Wis., Sept. 21-23.

INTERNATIONAL CITY MANAGERS ASSOCIATION, annual conference. International House, Chicago, Sept. 18-20.

NATIONAL SAFETY COUNCIL, 22nd Annual Safety Congress, Chicago, Oct. 2-6.

NATIONAL ASSOCIATION OF RAILWAY AND UTILITIES COMMISSIONERS, annual meeting, Cincinnati, Ohio, Oct. 10-12.

NEW ENGLAND WATER WORKS ASSOCIATION, annual convention, Bridgeport, Conn., Sept. 19-22.

Washington Notes

By Paul Wooton
Washington Correspondent

Allotment of $7,000,000 by the public works board for flood control on the lower Mississippi river will permit the army corps of engineers to resume work done by hand labor that was stopped several weeks ago by lack of funds. The allotment will not cover any new contracts.

In making the allotment the board assigned $2,200,000 to the Memphis district for completing suspended work on revetments and channel rectification to increase the discharge capacity of the river between Cairo and the head passes below New Orleans. The Vicksburg district will receive $3,550,000 and the New Orleans district $1,250,000. The board estimated that the total allotment will provide direct and indirect employment for 3,900 until next June.

One reason for delay on other federal projects for which allotments have previously been made was removed this week when Comptroller General McCarl finally approved warrants transferring the funds. Secretary Ickes stated that all federal agencies to which funds have been alloted will be expected to disburse the money as quickly as possible on the project schedules approved by the board. There has been a question whether the allotments should be spent one year or spread over a 2-year period.

The board is still mulling over the allotment of funds for river and harbor improvements. Another list of public building work will be approved shortly.

The appointment of engineers in the several states to handle local projects is expected to put this part of the program in motion throughout the country. There is no intention by the public works board to ease up on requirements that establish the eligibility of a municipality for assistance in financing.

Consideration is being given by Col. Henry M. Waite, Deputy Administrator, to making funds available for the elimination of grade crossings not embraced by the $400,000,000 road-building program but the conditions under which such work will be financed have not been decided.

Personal Notes

J. J. JESSUP, city engineer of Los Angeles, Calif., for the past four years, has resigned his office, effective Aug. 31. He will be succeeded by Lloyd Aldrich.

L. ROY BOWEN, formerly chief engineer of bridges and buildings, St. Louis, Mo., has opened an office for the general practice of engineering and architecture in the American Trust Building, St. Louis, Mo.

CLARENCE B. HOOVER, superintendent of waterworks, Columbus, Ohio, has been appointed consulting engineer for the city on the construction of a sewage disposal plant. He will relinquish his duties as waterworks superintendent, being succeeded in that position by his brother, Charles P. Hoover, now chemist in charge of the filtration plant.

JACOB L. CRANE, consulting engineer and city planner, Chicago, was appointed July 20 consultant to the housing division of the Public Works Administration. The next day Mr. Crane was asked to take

charge of regional planning and housing in the Tennessee Valley project. He has been identified as a zoning and planning expert with several Chicago suburbs and has had engagements in Colombia and Venezuela, South America, and in Russia, where he studied housing intensively as carried out by the Soviet government.

Obituary

ARCHIBALD S. FERGUSON, a Canadian railroad engineer, died at Port Arthur, Ont., at the age of 74.

RICHARD HENCKE, retired building contractor of St. Louis, Mo., died in that city on Aug. 2 after a long illness. He was 70 years old.

THOMAS M. OLIVER, contractor and architect of Utica, N. Y., died after a long illness on July 31 at the age of 72. He retired six years ago.

JAMES HORNER RICE, civil engineer of Little Rock, Ark., died Aug. 8 after an operation. He was 45 years old. Mr. Rice had served as engineer on a number of federal, state, and municipal construction projects.

MICHAEL H. DRINKWATER, for many years an engineer in the street department of the city of Boston, died at his home in Dorchester, Mass., Aug. 3, after a short illness. He entered the city's service in 1893 and retired last April on account of disability.

J. D. ROBERTSON, 52, deputy minister of public works for the province of Alberta, Canada, died on Aug. 3 at his home in Edmonton. Mr. Robertson was born in the east but went west in 1901 on Dominion land survey work. Moving to Edmonton when the province of Alberta was formed, he continued survey work there. In 1906 he became district engineer for the Provincial Works Department. Later he was appointed engineer of highways and some years after became deputy minister.

CHARLES A. HUTCHINSON, city engineer of Colton, Calif., for the past eleven years, died Aug. 6 after a paralytic stroke. He was 60 years old. He was chairman of the Lytle Creek Water Conservation Association and formerly was the Colton representative on the board of direcors of the Metropolitan Water District of Southern California.

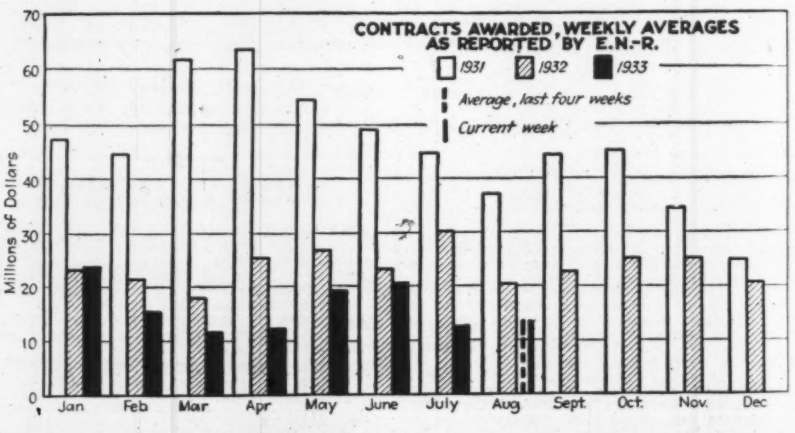
JOHN M. WATT, for more than 35 years a civilian engineer of the U. S. Engineer Corps, died in New Orleans on July 24 at the age of 73. His principal work was connected with lock and dam design and construction on the southern tributaries of the Ohio river. When the construction of the Panama Canal was taken over by the Engineer Corps he was appointed assistant division engineer of the Pacific division, remaining until the completion of the work. During the war he was put in charge of the Dallas, Texas, district, and for several years prior to his retirement was principal assistant engineer of the First New Orleans district.

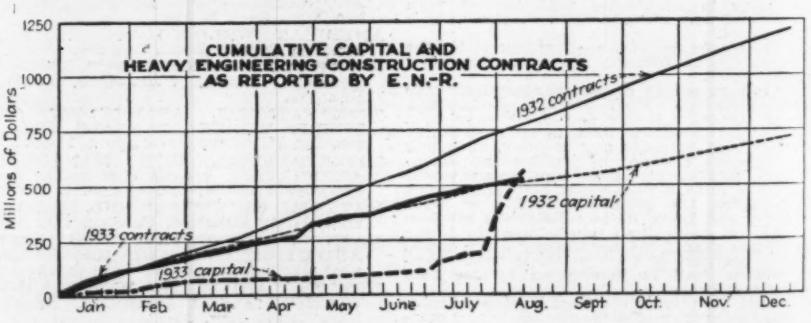
ELISHA LEE, vice-president of the Pennsylvania Railroad, died of apoplexy in New York City on Aug. 6. Mr. Lee was 62 years old. A few months after his graduation from Massachusetts Institute of Technology in 1892, Mr. Lee joined the Pennsylvania as a rodman in the office of the division engineer at Tyrone, Pa. In 1899 he was appointed assistant supervisor and in 1901 was named a supervisor. In 1905 he became assistant general manager of the Pennsylvania lines east of Pittsburgh. He was made an acting vice-president in 1917 and in 1923 was placed in charge of operation of the entire road. He held this post until September, 1926, when a new post, vice-president of the Pennsylvania, was creaed for him. During the war Mr. Lee was federal manager for the Pennsylvania's eastern lines.

Engineering Contracts and Capital

HEAVY engineering construction contracts reported the past week, totaling $13,464,000, barely rose above the average of the past four weeks, $13,045,000. For the second consecutive week private contracts have overtopped public works, awards in these classes amounting to $7,442,000 and $6,022,000, respectively. Federal work remains at a standstill, with only $272,000 reported in government contracts. Highway lettings are picking up, the past week's totals being $3,732,000, the highest in six weeks. With the federal allotments for highway work practically all made, road contracts are expected to increase rapidly from now on. Now that the state engineers have been appointed for the administration of the Public Works Act, early realization of many of the constantly growing list of proposed projects is expected.

Including $380,000,000 federal highway fund allotments, the total of new capital reported for the year is ahead of that of last year for the first time. Cumulative new capital to date amounts to $570,941,000, as compared with $506,800,000 for the same period of 1932. Only $36,000,000 in new private capital has been reported to date this year, mostly for breweries.





Contracts and Capital

CONSTRUCTION
(Thousands of Dollars)

	Weekly August, 1932	Average Four Weeks	Week Aug. 17, 1933
Federal Government..	$4,177	$336	$272
State and municipal...	12,440	5,360	5,750
Total public.....	$16,617	$5,696	$6,022
Total private.....	3,792	7,349	7,442
Week's total.....	$20,409	$13,045	$13,464
Cumulative to date:			
1932.....		$779,799	
1933.....		535,260	

NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

	Weekly August 1932	Average Four Weeks	Week Aug., 12, 1933
New capital issues....	$5,400	$98,923	$90,748
Cumulative to date:			
1932.....		$506,800	
1933.....		570,941	

ENR Cost and Volume Index

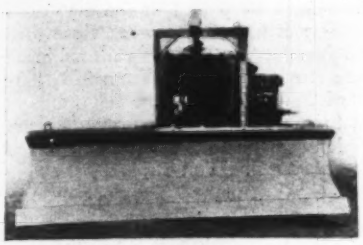
E.N.-R. Cost		E.N.-R. Volume	
August, 1933..	**167.00**	**July, 1933.....**	**65**
July, 1933.....	165.50	June, 1933.....	109
August, 1932..	156.80	July, 1932.....	166
1932(Average).	156.97	1932 (Average)...	127
1931(Average).	181.35	1931 (Average)...	220
1930(Average)	202.35	1930 (Average)...	260
1913 Average.....		100	

Construction Equipment and Materials

New Multi-Purpose Bulldozer

A new tool of the bulldozer type, which the maker claims can be used for many purposes where the ordinary bulldozer cannot function, is announced by R. G. Le Tourneau, Inc., Stockton, Calif.

The new tool is called an "Angledozer." The chief characteristics of the machine are its ease of adjustment to any angle of throw and any angle of blade pitch, its light weight, and its ability to work a



"Angledozer" by R. G. Le Tourneau

continuous line, without backing, in moving snow off a highway or pioneering a road along a hillside. The machine is extremely close-coupled, the conventional subframe being eliminated entirely. This brings the blade back very close to the front of the tractor, giving much better control of the blade and relieving front tracks and idlers from excessive strain.

Bituminous Distributors for Mounting on Light Chassis

A new series of bituminous distributors for mounting on a light truck chassis has been announced by the Austin-Western Road Machinery Co., Chicago, Ill.

This Junior line offers capacities of 400 500, and 600 gallons. The power unit is located in front of the tank, as in all Austin distributors, where it improves the load balance and is protected from dirt and oil. The pump and all other parts are placed so as not to hang below the truck chassis frame. This distributor may be mounted on any truck without special alterations, and the truck may be used with dump or other body when the distributor is not needed.

New bituminous distributor by Austin Western Road Machinery Co.

New Equipment in Brief

Turbine Pumps. A complete new line of turbine pumps has been announced by Roots-Connersville-Wilbraham, Connersville, Ind. Capacities range from 5 to 300 gal. per min. at heads up to 350 ft. Among the advantages claimed by the maker for this pump is a patented taper-edge impeller which gives the ability to vary the quantity while the pump is running. There is a slight increase in capacity as the head falls, with a noticeable decrease in power consumption.

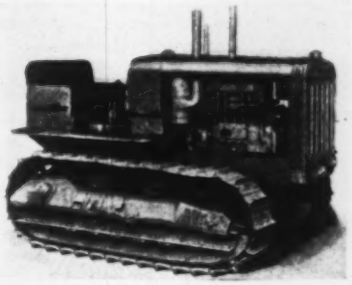
Oil Engine. The Waukesha Motor Co. have just concluded license arrangements with the Hesselman Co., Stockholm, Sweden, for the American rights under the Hesselman oil engine patents. Under these patents they will manufacture a line of heavy oil engines for truck, tractor, and general industrial service. Hesselman engines occupy a place midway between the automotive diesel engine and the gasoline engine. They burn the same fuels as a disel engine but use electric ignition.

Air Compressor. Schramm, Inc., West Chester, Pa., are announcing the addition of the Trac-Air compressor to their line of portable compressors and welders. The new compressor is especially designed for railroad maintenance and is built in size and dimensions that permit operation on the right of way without interfering with train schedules. The unit is of the crawler type and is adapted for traveling over rough and uneven surfaces and steep inclines. It is built in three sizes, 120, 240 and 360 cu.ft. displacements. Power is supplied by a heavy duty Buda engine.

Electric Welding. The Murex straight-gap method of electric welding has been announced by the Metal & Thermit Corp., 120 Broadway, New York, N. Y. In this method it is unnecessary to bevel the edges of plates to be welded. Plates are lined up with the welding edges parallel and a steel backing strip, $\frac{1}{8}$ in. thick or heavier, is used to close the bottom of the gap. The space between the plates varies according to the size of the electrode being used. An initial layer of weld metal to seal the lower edges of the gap is first laid down, using normal amperages. Then, with the current raised slightly, the weld is built up layer upon layer. The backing strip is then removed.

New Diesel Engine Tractor

The diesel 35 tractor has been announced by the Caterpillar Tractor Co., Peoria, Ill. The engine used differs only in horsepower and number of cylinders from the engines used in the Caterpillar diesel 75 and diesel 50 tractors. The Cater-



Caterpillar Diesel 35

pillar diesel 75 has six cylinders, the diesel 50 has four, and the diesel 35 has three cylinders, all manufactured on the same machines, employing the same parts.

The three-cylinder diesel engine used in this tractor is also available as a self-contained power unit.

Business Notes

BUCYRUS-ERIE CO., S. Milwaukee, Wis., announces that it has acquired the drill service of the Armstrong Mfg. Co. Effective immediately, Armstrong drills and bit sharpeners will be manufactured and controlled by the Bucyrus-Erie Co.

WOOD PRESERVING CORP., Koppers Bldg., Pittsburgh, Pa. have established an operating unit which will be supervised by Mr. Reamy Joyce and Mr. Sherman S. Watkins, formerly of the Joyce-Watkins Co., Chicago, Ill. The activities of Mr. Joyce and Mr. Watkins will be principally in connection with the Baltimore & Ohio R. R. cross-tie production and in the operation of the Green Springs, W. Va. treating plant.

New Publications

Bituminous Distributors. The Austin-Western Road Machinery Co., Chicago, Ill., has published bulletin 1283, 12 pages, which describes in detail the Austin bituminous distributors. The various parts of the distributors are described, as well as the auxiliary equipment.

Drafting Room Furniture. A new 60-page manual on drafting room furniture has just been issued by Hamilton Manufacturing Co., Two Rivers, Wis. The book is organized into nine sections, comprising oak plan files, steel plan files, the Hamilton-Calumet line, pedestal-type drawing tables, 4-post type drawing tables, combination drawing and reference tables, drawing boards, drafting room accessories, and artists' equipment.

Welding Stainless Alloys. An 18-page booklet entitled "The Welding of Enduro Stainless Alloys" has just been published by the Republic Steel Corp., Central Alloy District, Massillon, Ohio. The booklet contains information on the various welding methods as applied to different types of Enduro stainless alloys including electric arc welding, gas welding, spot and projection welding, steam welding and flash welding.

Magnetic Inspection. The Ferrous Magnetic Corp., 110 Washington St., New York, N. Y., has published a 20-page catalog describing and illustrating the Fermango magnetic inspection equipment for wire and wire rods and strip steel. The inspection method used is to compare the magnetic permeability of a standard sample and the material being tested. The equipment will inspect and detect the following: hardness, grain size, stress, carbon content, carbide segregations, inclusions, surface decarbonization, graphitic carbon, normal and abnormal steel and heat treatments.

Current Construction Unit Prices

Earth Dam for Hartford Water Supply

CONTRACT was awarded on July 21 by the Water Bureau of the Metropolitan District Commission of Hartford County, Conn., for the construction of the stream control works and the lower portion of Bills Brook earth dam, the start of the third step in Hartford's water supply system. This structure is the beginning of construction on an extensive project for additional water supply. The dam, located on the East Branch of the Farmington River, 20 miles north of the city, will have an ultimate height of 130 ft. and length of 2,000 ft. It will be a rolled-earth fill with a concrete core wall. The reservoir will cover 4 sq.mi. and have a capacity of 30 billion gal. A bond issue of $8,600,000 was approved some time ago to finance the project. The dam will be built to only a third of its ultimate height under the present contract, and will be raised as necessity develops for more storage capacity.

Forty inquiries were made for plans, and eighteen bids were submitted, ranging from $538,755 (contract) to $895,280. Caleb Mills Saville is general manager and chief engineer of the Water Bureau of the Commission. Following are complete unit prices of the three low bidders. (A) C. & R. Const. Co., Boston, contract; (B) Perini & Son, Inc., Framingham, Mass.; (C) Geo. M. Brewster & Son, Inc., Bogota, N. J. There was less than $9,000 difference between the low and second bidders.

BILLS BROOK DAM AND STREAM CONTROL WORKS

Item	Quan.	Unit Prices A	B	C
1. Clearing and grubbing, acres	10	$50.00	$60.00	$300.00
2. Soil removal, c.y.	27,000	.30	.20	.58
3. Stream control during const.	lump	1,000	1,000	14,000
4. Earth excavation, c.y.	120,000	.40	.38	.29
5. Rock excavation, c.y.	40,000	1.00	1.75	1.17
6. Pumping	lump	500.00	3,000.00	1,567.00
7. Special preparation rock surfaces, sq.yd.	4,000	.50	.25	.09
8. Earth fill, cofferdams, c.y.	25,000	.40	.33	.06
9. Rock fill, cofferdams, c.y.	36,000	.30	.23	.04
10. Riprap, c.y.	2,700	1.00	2.00	1.90
11. Earth fill, rolled 12-in. layers, c.y.	350,000	.30	.20	.23
12. Earth fill, rolled 8-in. layers, c.y.	85,000	.30	.24	.29
13. Soil core and blanket, c.y.	75,000	.35	.50	.46
14. Selected fill, rolled 6-in. layers, c.y.	20,000	.35	.33	.35
15. Topsoil, dressing and seeding, c.y.	6,000	.75	.65	.70
16. Tile drains, 8 in. and under, l.f.	2,500	.35	.75	.30
17. Tile drains, 10 and 15 in., l.f.	3,700	.75	1.00	.82
18. Catch basins, each	9	70.00	80.00	95.00
19. Paving, cy.yd.	1,500	1.50	5.50	4.67
20. Concrete, core wall, c.y.	16,500	4.50	3.89	4.77
21. Concrete, diversion conduit, c.y.	3,200	5.80	6.66	5.68
22. Concrete, gate houses, c.y.	1,800	7.00	7.92	5.58
23. Concrete, retaining walls, c.y.	3,400	4.90	6.36	5.24
24. Concrete, channel lining, c.y.	1,350	5.00	7.08	5.10
25. Drill holes in rock, l.f.	5,000	.20	.40	.23
26. Grout, c.y.	100	8.00	35.00	8.68
27. Timber structures, M. f.b.m.	85	60.00	70.00	83.00
28. Portland cement, bbl.	37,000	2.30	2.39	2.39
29. Broken stone, c.y.	100	3.00	2.00	2.80
30. Metal, reinf. and water stpps., lb.	350,000	.0325	.045	.035
31. Misc. metal, lb.	16,000	.08	.12	.08
32. Waterproofing, sq.ft.	9,000	.40	.20	.12
33. Cleaning up	lump	3,000.00	1,000.00	600.00
Total		$538,755	$549,260	$567,700

Piers, Little Bay Bridge, New Hampshire

AN INTERESTING set of bids was received by the New Hampshire Toll Bridge Commission for the construction of the piers and abutments for the Little Bay highway bridge between Portsmouth and Dover, N. H. The original proposal asked for bids in the usual manner. Later supplementary proposals were sent to bidders asking for a lump sum designation of increased cost if the project should be financed by federal funds under NIRA, which involves regulation as to hours of work, use of local labor and payment of prevailing wages. The ten bidders submitted additions to their regular proposals ranging from 1.8 per cent to 9.9 per cent above the original bids. The average increase was 5.8 per cent. Alternate bids were received on completion dates of March 31, 1934, and December 30, 1933. The increase for the earlier completion amounted to about 3 per cent in most bids. Contract was awarded on the basis of completion December 30, 1933.

The substructure consists of eight piers and two abutments. The location is at the mouth of Little Bay, where the bay joins the Piscataqua River. The site is subject to swift tidal currents. The maximum depth of excavation expected is about 30 ft. below mean low tide. All of the piers will be faced with New Hampshire granite (specified) from below low water line to the top. New Hampshire products are specified for coarse aggregates. The four main piers are 55x14 ft. in plan at the base. The other four are 38x12 ft. in size. All concrete must be placed in the dry, calling for tight cofferdams. The B.&M. Ry. crosses close to the site, and the existing highway crosses on the railroad bridge.

Ten formal bids were received, ranging from $238,400 to $530,048 on the basis of March completion. Contract was awarded for $244,400 to the low bidder on the December completion basis. If federal financing is received, the amount of contract will be increased. The Toll Bridge Commission consists of Governor J. G. Winant and six more members, including Federic E. Everett, state highway commissioner. Fay, Spofford & Thorndike, Boston, are consulting engineers. Following are complete unit prices of the three low bidders. (A) Crandall Engineering Co., Cambridge, Mass., contract; (B) Kennebec Const. Co., Waterville, Me.; (C) Blakeslee-Rollins Corp., Boston.

SUBSTRUCTURE, LITTLE BAY BRIDGE, NEW HAMPSHIRE

Item	Quan.	Unit Prices A	B	C
1. Concrete, abutments, c.y.	550	$21.00	$20.00	$15.00
2. Reinf. steel, lb	30,000	.05	.04	.04
3. Excav., abutments, c.y.	480	5.00	1.30	1.00
4. Embankment, abutments, c.y.	4,800	.60	.80	.50
5. Riprap, abutments, c.y.	450	8.00	2.60	2.00
6. Eight piers, complete	lump	207,110.00	225,000.00	248,615.00
7. Additional depth, small piers, ft.	8	410.00	350.00	400.00
8. Additional depth, main piers, ft.	8	760.00	500.00	400.00
9. Total, compl. March 31, 1934	....	$238,400*	$249,634	$268,245
10. Additional for completion Dec. 30, 1933	....	6,000.00*	10,000.00	No bid
11. Additional for completion March 31, 1933 under NIRA (add to Item 9)	....	18,000.00	20,000.00	10,000.00
12. Additional for completion Dec. 30, 1933 under NIRA (add to Items 9 and 10)	....	18,000.00	20,000.00	No bid

*Contract.

Highway Grading and Drainage, Delaware

THE STATE HIGHWAY DEPARTMENT of Delaware late in July let contracts for five road grading, drainage and slag surfacing projects. Contract unit prices of the principal items are given in the following table. The projects, contractors, and contract prices are as follows: (A) Porters-Salem Church project, 4.284 miles, A. Petrillo & Co., Wilmington, $25,061; (B) Todds Chapel-Prospect Church project, 5.7 miles, George & Lynch, Dover, Del., $21,207; (C) Holladsville-Maryland Line project, 4.037 miles, George & Lynch, Dover, $14,267; (D) Zoar Camp-Angola project, 6.9 miles, Highway Eng'g. & Const. Co., Selbyville, Del., $20,450; (E) Coverdale Cross Road-Smith School project, 3.7 miles, Old Line Const. Co., Chestertown, Md., $9,752. W. W. Mack is state highway engineer.

DRAINAGE AND GRADING, DELAWARE HIGHWAYS

Item	Project and Unit Prices A	B	C	D	E
Grading, ave. per mile	$351	$407	$399	$274	$250
Borrow, per cu.yd.	.80	.45	.40	.40	.50
Slag surface, per ton	2.80	3.08	3.24	3.25	2.90
Corr. met. pipe, 10 in., per ft.	1.29	....	....	....	....
Corr. met. pipe, 15 in., per ft.	1.75	....	....	....	....
Corr. met. pipe, 18 in., per ft.	2.00	....	....	....	....
Reinf. Conc. pipe, 15 in., per ft.	1.25	1.00	1.00	1.00	1.00
Reinf., conc. pipe, 18 in., per ft.	1.50	1.20	1.40	1.50	1.30
Reinf. conc. pipe, 24 in., per ft.	2.25	2.50	2.25	2.25	2.25
Reinf. conc. pipe, 30 in., per ft.	4.00	3.00	3.00	....	....